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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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STORAGE REQUIRED FOR THE REGULATION OF STREAM FLOW

By Charles E. Sudler,* M. Am. Soc. C. E.

Synopsis

This paper presents a study followed by a set of curves for determining the dependable stream flow to be expected due to a given reservoir capacity, founded on a paper entitled "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply", by Allen Hazen, M. Am. Soc. C. E. Having found through their use in literally hundreds of instances, that Mr. Hazen's "seasonal storage curves" can be depended upon, the writer has developed extensions to them, believed to be equally reliable, for those high rates of use or draft not covered by these "seasonal" curves.

The paper also discusses those factors due to which a reservoir located on a tributary, or at a point some distance up stream from the point of use, may be less useful than one having a point of use corresponding with the dam site; and develops a series of curves for predicting the dependable flow at the point of use in all such instances. The percentage of time during which a regulating reservoir may remain only partly full, and the length of continuous periods of partial depletion of storage, are also discussed, and diagrams are presented to solve these problems.

A number of computations are made, assuming storage capacities and rates of use as determined by the diagrams, and applying them to various streams for which flow records covering a period of years were available. These fully attest the reliability of the curves.

The New York Water Power Investigation has completed a thorough study of the water power possibilities in New York State, in the course of which it fell to the writer to study the hydrology of the various streams and the regulating effect of such reservoirs as could be found on them.

In the simplest case, involving only one practicable reservoir site, the problem was to determine its most economical capacity and the dependable flow which it would insure at the dam site. In other instances a number of good storage possibilities were found and many scattered power sites existed on the same water-shed. A further complication lay in the fact that the run-off at some power sites greatly exceeded that at the reservoirs. As the most economical capacity could only be found after comparing the regulated flows

Note.—Written discussion on this paper will be closed in March, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

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due to various capacities, through the heads affected, it was obvious that attack by the usual methods would have involved a tremendous amount of detail work. It was also realized that consistent results could not be expected where small changes in reservoir capacity or in point of regulation were involved, if the study were to depend on the ordinary use of the comparatively short-term flow records available.

The paper by Allen Hazen, M. Am. Soc. C. E., on "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply",* offered a promising tool for effecting this wholesale regulation study within the limited time available, and proved to be of the greatest value. The dependable flows indicated by Hazen's curves were checked by many direct studies of discharge records, with gratifying results. It was found that these curves gave consistent relative solutions, whereas the older method was most unsatisfactory, owing to insufficient data.

Hazen's application of the two easily derived constants, the coefficient of variation of annual flow and the storage constant, provides a simple and reliable means of ascertaining the possibilities of regulation for moderate rates of draft. The writer, in this paper, formulates a set of diagrams that are believed to be more nearly correct than those of Mr. Hazen for high rates of draft, involving large quantities of storage. Although the treatment in deriving these curves in many respects follows that of Mr. Hazen, it differs in some important details. The paper presents a discussion of the length of time and percentage of time that a reservoir will remain less than full, and a study of the usefulness of storage on a tributary.

General Considerations

The merits of the method of study here given are generally unrecognized. Few of the many reports and studies of hydro-electric development mention the dependability of the proposed regulation. In most of them the matter is dismissed by stating in substance that the proposed use or draft represents the minimum flow during some period for which actual or representative hydrographs were available.

The usefulness of storage is frequently determined from a mass curve by plotting the flow to be obtained from using all the storage capacity each year. The resulting flows are then drawn as a regulated "flow-percentage of time" curve, and it is assumed that after regulation similar flow will result. To the writer such a method does not appear trustworthy: First, because the original record of flow is usually too short to portray all typical future conditions; and, second, as the flow is not known in advance it is impossible to release the storage so as to maintain steady flow without either surplus or deficiency at the end of the season. It would seem more desirable to adopt a method such as Hazen offers, by which a dependable flow is assured for some percentage of all future years, such as 90 or 95% of the time.

Run-off, deficiency, etc., are generally referred to in the following study as types which express the probability of their occurrence. Thus "90% year

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

run-off" is that value which is exceeded during 90% of the time; "90% year seasonal storage" is that quantity which will be sufficient during 90% of the time; and "90% regulation" refers to that flow which can be dependably maintained during 90 out of every 100 years. The average annual stream flow at the point under examination is taken as unity, and drafts, etc., are expressed as fractions of this average stream flow.

Types of Regulation

Impounding reservoirs may be used in two general ways. One assumes that the storage capacity provided will be sufficient to maintain not less than some computed outflow or "draft" over a certain percentage of future years. Such regulation takes cognizance of the fact that years of low annual run-off will occur, and provides storage, if required, to make up any deficiencies during these dry years. A part of the storage may be used only occasionally; on the other hand, the reservoir may not be full throughout a period of years.

The other type, frequently used for power purposes, particularly where small storage is available, does not attempt to maintain a given minimum flow at all times, but rather to utilize all, or nearly all, the storage each year, so as to secure the maximum possible energy available with the installation.

Regulation as here contemplated provides for the release of stored water whenever the natural flow falls below an assumed rate of use, or draft, although in practical operation it is possible to operate the storage so as to secure, at times, additional flow without sacrificing the dependability of the assumed draft or minimum regulated flow.

Regulation for water supply or power purposes may serve incidentally to reduce flood flows, but unless the reservoirs provided are exceptionally large, or are operated strictly for that purpose, they do not insure complete freedom from destructive floods.

Seasonal, Annual, and Monthly Storage

Seasonal storage is a term used to measure the deficiency of flow in respect to a given rate of draft or use during some one year. It represents the quantity of water which must be released during the year to maintain the assumed draft. It does not necessarily indicate the reservoir capacity required, as may be seen in Fig. 1. In this diagram the seasonal storage required for each of the four years is shown by the ordinates *a*, *b*, *c*, and *d*, respectively, while the total storage needed for these four years is equal to *c* plus *e*.

If, during the period for which regulation is provided, the total flow of one or more years is less than the assumed draft, seasonal storage alone may be insufficient, and additional capacity, to be filled during wet years, must be provided to make good the deficiencies of the very dry years. In Fig. 1 this is shown by *e* which measures the annual deficiency of the first two years. This is called "annual" storage or deficiency.

"Monthly" storage is an expression herein used to define seasonal storage minus annual deficiency for any typical year. In Fig. 1 for the third

year, the seasonal storage equals *c*, the annual deficiency is *f*, and the monthly storage is *c* minus *f*.

Considerations of Annual Stream Flow and Storage

Applying the theory of probabilities to the study of stream flow Mr. Hazen found it possible to classify different streams according to their habits of annual fluctuation, using an appropriate coefficient of variation (*C V*) of annual flow to express the type into which any stream would fall. Stream flow of the type represented by this *C V* might be expected to fluctuate between limits which, for any given period, increase with the increase of *C V*. Using an appropriate "run-off-percentage of time" curve, as based on the computed *C V*, the theoretical run-off of all the years of a stated period may be ascertained, and, by chance selection of these annual values, an artificial record may be constructed.

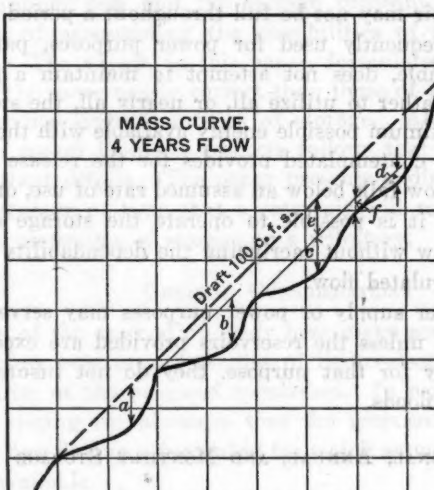


Fig. 1.

It is thought that a long artificial record built up in this way offers a much more satisfactory means of studying annual deficiencies than such generally rather short-flow records as are available for American streams. When high rates of draft are considered, the deficiencies occasioned by a sequence of dry years are of great importance, but a flow record of even 50 years' duration does not completely cover the range and frequency of such deficiencies. For example, suppose the fluctuations of annual run-off of a stream for 10 years could be measured by ten definite values. The number of possible arrangements of these ten values is so great that a period of 36 288 000 years might elapse before any particular sequence of 10 years would be repeated!

Having assumed an "annual run-off-percentage of time" curve appropriate for the stream as expressed by its *C V*, an artificial record of, say, 1 000

years' length may be built up by dividing the curve into 1 000 parts and selecting the values read at the middle of each part. A sequence is obtained by drawing the numbers representing the annual run-off values by some chance method. In the present instance fifty annual run-off values were selected and put on cards, which were shuffled and drawn one by one until the whole fifty were used. Twenty such drawings were made, thus providing a 1 000-year artificial record.

Computation of Annual Storage

In order to ascertain the probable annual storage requirements the first method tried assumed a reservoir of infinite capacity, full at the start; then, with various drafts, examination was made as to the depletion at the end of each year. These depletions were then plotted in order of magnitude as in Fig. 2.

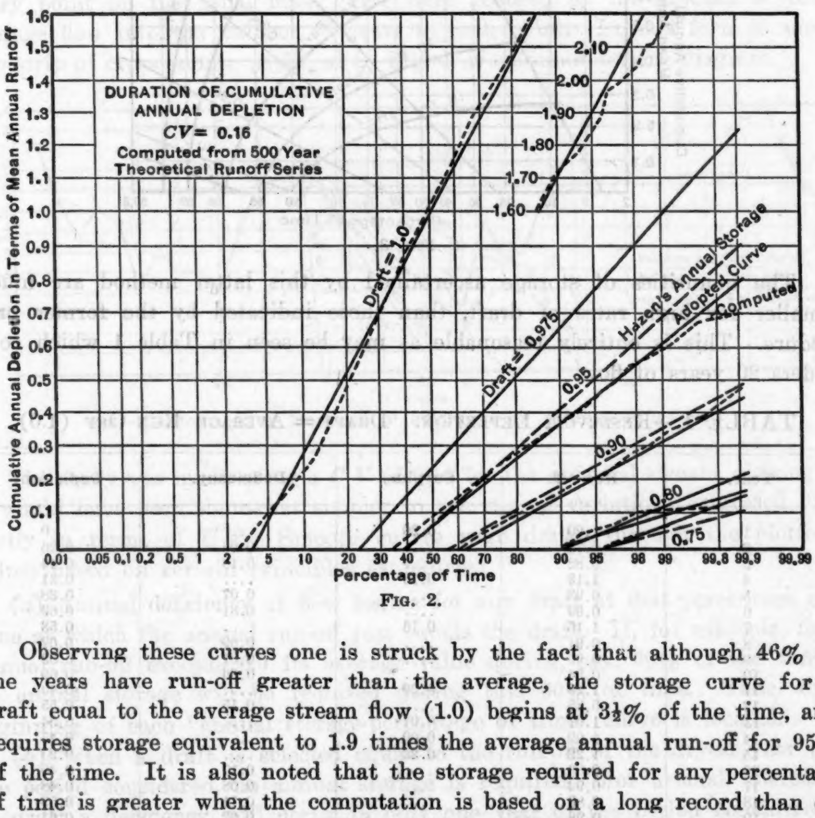


Fig. 2.

Observing these curves one is struck by the fact that although 46% of the years have run-off greater than the average, the storage curve for a draft equal to the average stream flow (1.0) begins at 3½% of the time, and requires storage equivalent to 1.9 times the average annual run-off for 95% of the time. It is also noted that the storage required for any percentage of time is greater when the computation is based on a long record than on a short one. This is shown in Fig. 3 which embodies the same storage curves, with the addition of lines intercepting the quantities required for 5, 10, 20, 50, and 100-year periods. These values were obtained by computing the maximum storage required for each draft, for each period of 5, 10, 20 years, etc.,

and taking the average of these maximum values as the most probable requirement.

The inconsistencies thus seen led to examination of the artificial record with the assumption of definite storage values for the various drafts, thus approximating actual regulation. For each draft a number of reservoir capacities were assumed, and with each capacity the years of deficiency following the use of the storage, were noted.

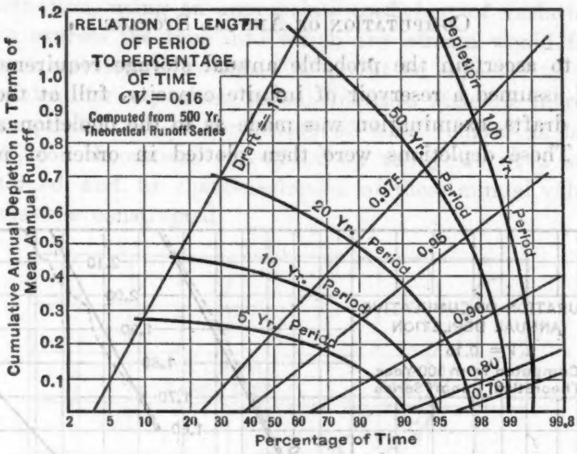


FIG. 3.

The quantities of storage ascertained by this latter method are much smaller for high rates of draft, than those indicated by the former procedure. This is entirely reasonable as may be seen in Table 1 which considers 20 years of flow.

TABLE 1.—RESERVOIR DEPLETION. DRAFT = AVERAGE RUN-OFF (1.0).

Year.	Run-off.	Surplus.	Deficiency.	Depletion.
1	1.20	0.20	....	0
2	0.74	....	0.26	0.26
3	0.82	....	0.18	0.44
4	1.13	0.13	....	0.31
5	0.93	....	0.07	0.38
6	0.69	....	0.31	0.69
7	1.16	0.16	....	0.53
8	0.91	....	0.09	0.62
9	0.99	....	0.01	0.63
10	0.94	....	0.06	0.69
11	1.29	0.29	....	0.40
12	0.85	....	0.15	0.55
13	1.01	0.01	....	0.54
14	1.09	0.09	....	0.45
15	1.22	0.22	....	0.23
16	0.83	....	0.17	0.40
17	0.92	....	0.08	0.48
18	0.89	....	0.11	0.59
19	0.98	....	0.02	0.61
20	1.06	0.06	....	0.55

By the first method, assuming a reservoir of infinite capacity, the storage for 19 out of 20 years would be 0.69, which is the nineteenth largest

value of depletion. If, however, a reservoir with a capacity of 0.44 be assumed, it is seen that the draft of 1.0 can be maintained during the first 5 years, with a deficiency of 0.25 in the sixth year. In the seventh year 0.16 of water is in the reservoir, which is sufficient to carry the draft over the eighth, ninth, and tenth dry years. In the eleventh year the reservoir is refilled to the extent of 0.29 and by the end of the fifteenth year, it is completely refilled, and is never thereafter entirely emptied. Actually, then, storage of 0.44 is sufficient for 19 out of this 20 years.

Graphical Use of Artificial Record—The mass curve offers a convenient means of studying storage as the cumulative flow at every point of time is shown graphically. For this study instead of plotting the accumulated flow at the end of every year, the accumulated surplus or deficiency relative to the average run-off, was used. The result is the same as if the values at every point on the usual mass curve were reduced by the product of the average flow into the number of years to each point. In this form a narrow strip of cross-section paper, as in Fig. 4, is sufficient for the diagram.

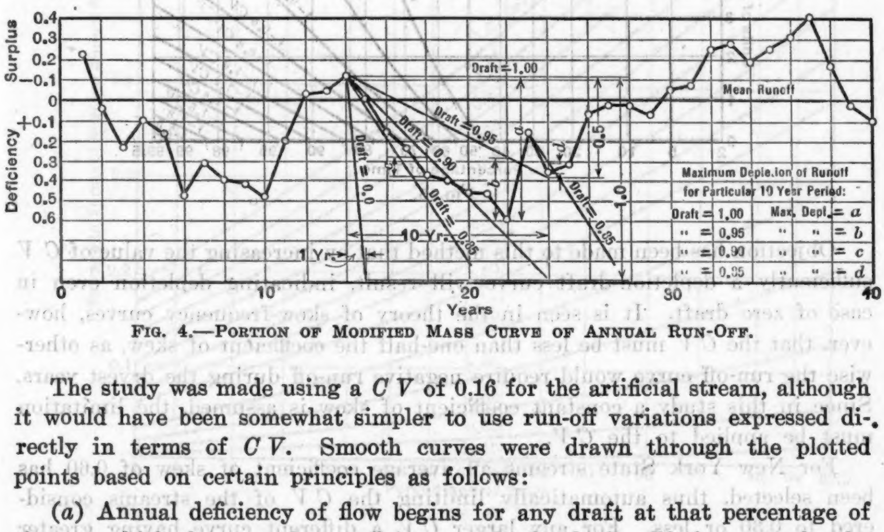


FIG. 4.—PORTION OF MODIFIED MASS CURVE OF ANNUAL RUN-OFF.

The study was made using a $C\,V$ of 0.16 for the artificial stream, although it would have been somewhat simpler to use run-off variations expressed directly in terms of $C\,V$. Smooth curves were drawn through the plotted points based on certain principles as follows:

(*a*) Annual deficiency of flow begins for any draft at that percentage of time at which the annual run-off just equals the draft. If, for example, the annual run-off exceeds 0.9 its average value during, say, 60% of the time, no annual storage will be required during this 60% of time. Thus, the beginning of each "annual storage-percentage of time" curve is located.

(*b*) When a draft is selected equal to the run-off of the dryest year of the period considered, no annual storage is required. For a small increase in draft a deficiency will occur in only one year of the period considered, and as long as this condition holds for a given increase in draft there must be an equal increase in storage. If the storage is plotted against the draft, both to the same scale, the curve must have a minimum slope of 45 degrees. The actual storage curve must then begin tangent to this 45° slope and diverge upwardly from it.

Relation of $C\,V$ to Annual Storage

The annual storage computations were made for a theoretical stream the $C\,V$ of which was 0.16. Hazen has shown that if storage is expressed in terms of $C\,V$ and drafts are taken as 1—($C\,V \times$ a constant), then for any given storage and draft, the draft will always be maintained through the same percentage of years and the deficiencies will be directly proportional to $C\,V$. Hence, the results obtained with the 1 000-year record can be reduced to terms of $C\,V$, as shown in Fig. 5.

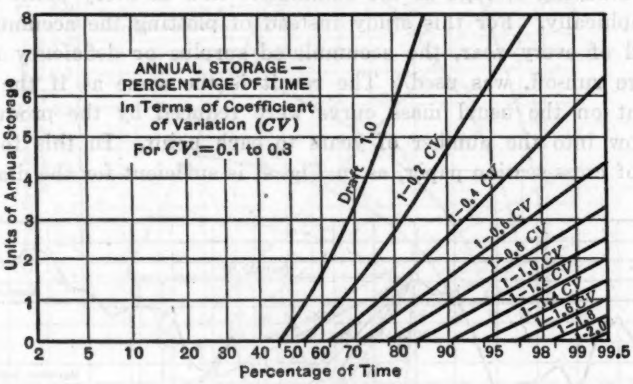


Fig. 5.

Objection has been made to this method that by increasing the value of $C\,V$ sufficiently a depletion-draft curve will result, indicating depletion even in case of zero draft. It is seen in the theory of skew-frequency curves, however, that the $C\,V$ must be less than one-half the coefficient of skew, as otherwise the run-off curve would require negative run-off during the dryest years. Since in this study a constant coefficient of skew is assumed, the limitation must be applied to the $C\,V$.

For New York State streams an average coefficient of skew of 0.60 has been selected, thus automatically limiting the $C\,V$ of the streams considered to 0.30 or less. For any larger $C\,V$ a different curve having greater skew must be adopted.

Computation of Seasonal Storage

Seasonal storage is a measure of the deficiency in respect to a given rate of flow during single years. Hazen, in his study, which embraced the flow of streams in various sections of the United States, found, at least as far as was disclosed by the considerable amount of data at hand, that his "seasonal storage—percentage of time" and "seasonal storage—draft" curves could be applied to all streams, even to those whose characteristics of flow as shown by annual "storage-percentage of time" curves differed widely. For example, assuming a draft equal to 0.5 of the average flow, the

Gunpowder River required an average storage equal to 5 days' flow, and 95% of time storage equal to 72 days' flow. The South Platte River required an average storage of 60 days' flow and 95% storage equal to 127 days' flow. In both cases the difference between the average and 95% storage is 67 days' flow at the assumed draft. The 95%-year annual run-off of the South Platte River is much less than that of the Gunpowder River, but this does not affect the relative differences between the average and the 95% requirements as expressed in day's flow at drafts that are equal proportions of the average run-off in both instances.

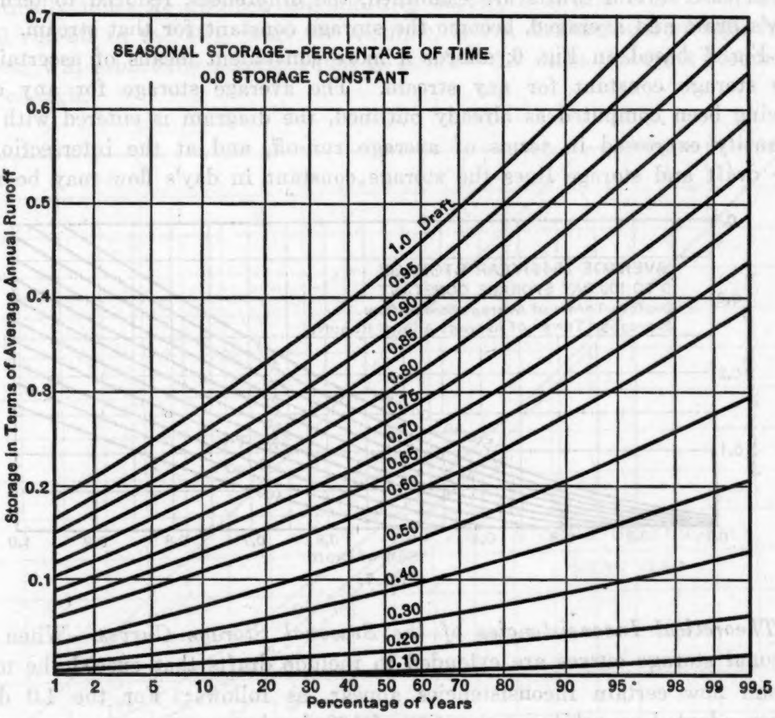


FIG. 6.

Expressing seasonal storage in terms of day's flow at the assumed rate of use or draft, Hazen found that a definite relation between various drafts holds for all streams; also that a definite number of days' draft would represent the difference between the seasonal storage for the average year and that for any other year. Based on these relations he computed the number of days seasonal storage required for the mean year, and the 95% year, for various rates of draft, and connected these by skew probability curves that appeared to represent the data. The actual values given these curves are greater than those of any of the streams examined, but the differences between any two rates of draft, or between the storage for one percentage of time and another percentage of time, are directly applicable to any stream.

Thus, by applying a suitable constant number of days' draft, actual or definite values can be taken from the curves.

"Seasonal storage-percentage of time" curves based on Hazen's mean and 95%-year values,* connected by suitable skew probability curves are shown on Fig. 6. To apply these to a particular stream for which a few years of flow record are available, the actual average seasonal storage for one or more rates of draft is computed from the flow record in terms of the mean annual run-off. These values are compared with those indicated at the mean or 54%-year on Fig. 6, for similar drafts, and the difference, or in case several drafts are examined, the differences, reduced to terms of day's draft and averaged, become the storage constant for that stream.

Fig. 7 based on Fig. 6, affords a more convenient means of ascertaining the storage constant for any stream. The average storage for any draft having been computed as already outlined, the diagram is entered with this quantity expressed in terms of average run-off, and at the intersection of the draft and storage lines the storage constant in day's flow may be read.

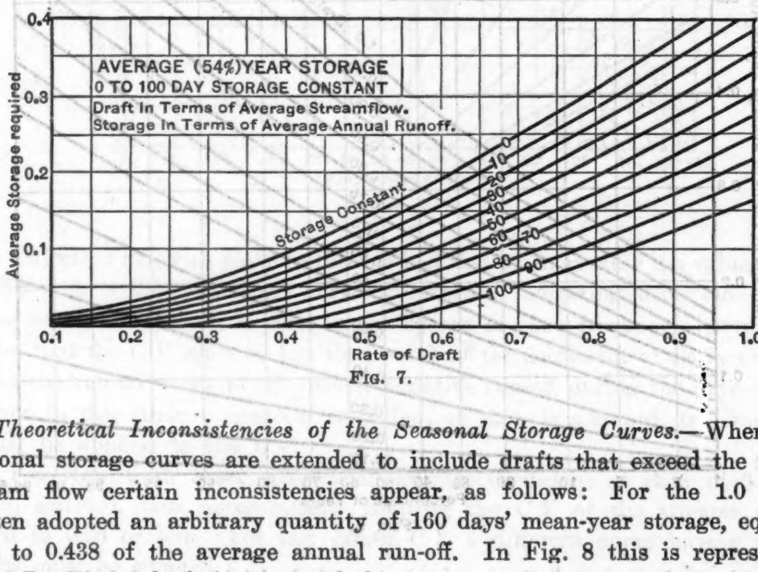


FIG. 7.

Theoretical Inconsistencies of the Seasonal Storage Curves.—When the seasonal storage curves are extended to include drafts that exceed the mean stream flow certain inconsistencies appear, as follows: For the 1.0 draft Hazen adopted an arbitrary quantity of 160 days' mean-year storage, equivalent to 0.438 of the average annual run-off. In Fig. 8 this is represented by *A B*. If the draft is increased the storage must increase also. At some draft, equal to the highest rate of run-off during the year, the seasonal storage required will be equal to the draft minus the annual run-off (Draft—1), and this relation will hold for all higher drafts. That is, the storage curve will eventually become tangent to the straight line, *B C*, which has a slope of 45 degrees. The complete storage curve for 0-day constant will then be the line, *O A C*. For some other stream having a 56-day constant, for example, Hazen's curve will be *O E H*. This is obtained by subtracting 56 days' storage from the 0-day curve, for each draft. This curve, *O E H*, evidently cannot become tangent to the line, *B C*, although theoretically it should

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1583, Table 8.

do so, as the average annual run-off is still unity as before. If the curves are to be extended so as to apply to drafts in excess of the average run-off, a different formula must be used, and they must become tangent to the line, *B C*, somewhat as shown by the curve, *O E F*.

Theoretically, a stream might require more storage than is indicated by the 0-day constant. It would then have a negative storage constant. The extreme case would be a stream requiring storage equal to a draft for all rates of draft. For a draft of 1.0, 365 days' storage would be necessary, and the storage constant would become —205 days. The storage curve for such a stream would be the straight line, *O D* (Fig. 8). All storage draft curves theoretically must lie between the limiting curves, *O D* and *O B C*. If the annual run-off in the 95%-year, for example, is 0.8, the seasonal storage for this year will eventually become equal to Draft — 0.8 when the draft becomes large enough.

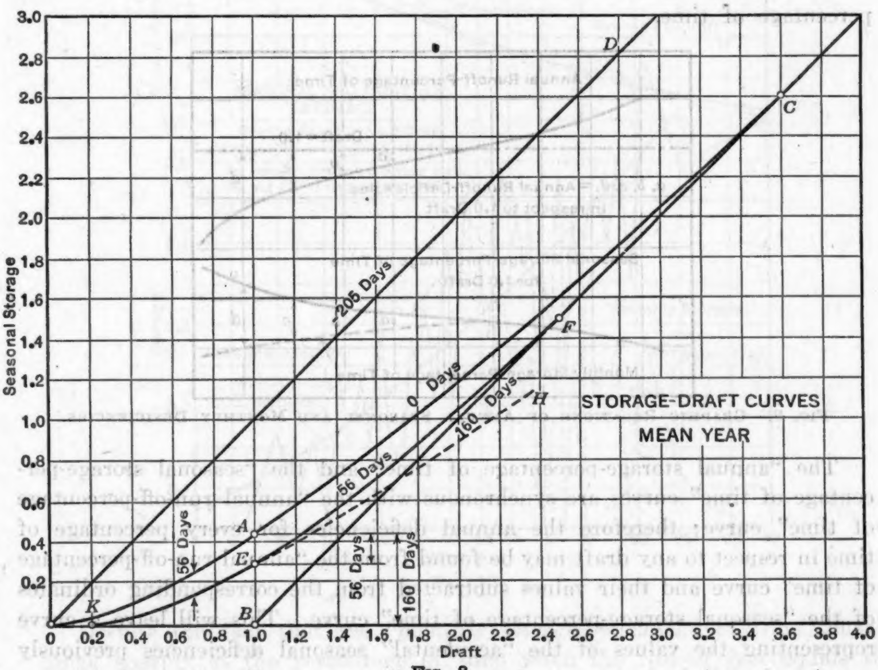


FIG. 8.

COMBINING SEASONAL AND ANNUAL STORAGE

Seasonal storage represents the total deficiency of flow of single years. If the flow for the year is less than the draft there will be an annual deficiency. The draft cannot be maintained unless the annual deficiencies are provided for by a reservoir capacity equal to the combined deficiencies of the dryest series of years expected, plus a certain quantity of additional storage which may be used and again replaced year after year. This addi-

tional quantity will be given the name "monthly" storage. Its magnitude, to combine with annual storage for various drafts, is to be found.

Examination shows that although the seasonal storage required to maintain a given draft may vary considerably for two yearly periods having the same run-off, nevertheless, as a general principle, the required seasonal storage is least for years of highest run-off, and increases as the annual run-off decreases. Thus, it may be said that the seasonal "storage-percentage of time" curve is made up of two factors; one due to a decreasing annual run-off as a longer percentage of time is considered, and measuring the annual deficiency of run-off; and the other due to what might be called the accidental fluctuations within the dryer season of the year. Since, however, the annual deficiency has been studied separately, and provision made for it by the use of annual storage, it should not again be included as would be necessary were the total storage for a given percentage of time to be assumed as equal to the sum of annual plus the seasonal storage, for that percentage of time.

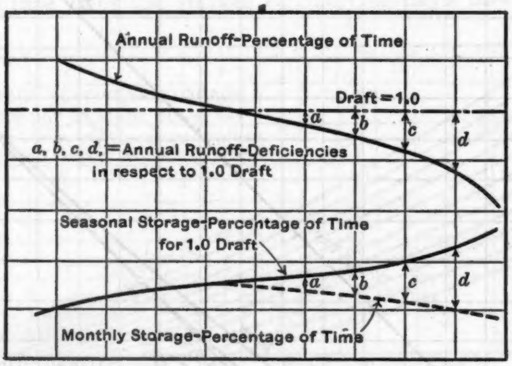


FIG. 9.—GRAPHIC RELATIONS OF ANNUAL SEASONAL AND MONTHLY DEFICIENCIES.

The "annual storage-percentage of time" and the "seasonal storage-percentage of time" curves are synchronous with the "annual run-off-percentage of time" curve; therefore the annual deficiencies for every percentage of time in respect to any draft may be found from the "annual run-off-percentage of time" curve and their values subtracted from the corresponding ordinates of the "seasonal storage-percentage of time" curve. This will leave a curve representing the values of the "accidental" seasonal deficiencies previously mentioned, called the curve of "monthly" storage. The "annual storage-percentage of time" curve, when superposed on this "monthly storage-percentage of time" curve, indicates the total storage required. Fig. 9 shows an "annual run-off-percentage of time curve" with deficiencies in respect to a draft of 1.0; and below, a "seasonal storage-percentage of time" curve for this draft, with the several deficiencies to be subtracted, leaving the so-called "monthly" storage curve. This follows the seasonal storage curve up to that percentage of time for which the run-off exactly equals the draft, then breaks and falls below the seasonal storage curve, as shown.

Fig. 10 shows the application of the principle to the Croton River. The actual annual run-off for fifty-two years of record is set in descending order of magnitude alongside the theoretical "run-off-percentage of time" curve for such a stream, as measured by its $C\,V$. The actual seasonal storage for years corresponding with the run-off as shown on the upper curve is indicated in Fig. 10, as also are the "monthly" storage values obtained by subtracting the actual annual deficiencies from the seasonal storage. The corresponding theoretical curves are also drawn. The "accidental" fluctuations of seasonal storage for years of nearly the same run-off are clearly evident, and the theoretical curves seem to represent the means of these fluctuations fairly well.

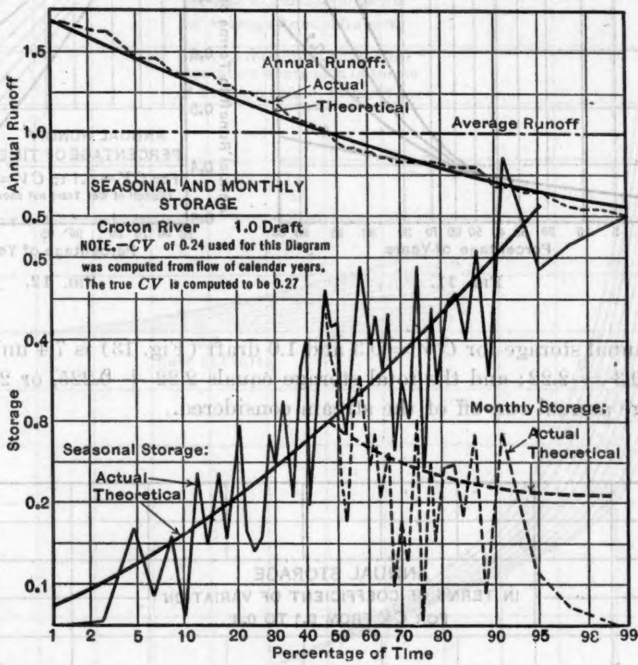


FIG. 10.

Despite the fact that the "monthly storage-percentage of time" curves break sharply at the percentage of time when the run-off just equals the draft, the curves of total storage as obtained by adding the annual storage to the monthly storage values for identical percentages of time, are fair throughout their length, as should no doubt be the case. Where large draft is considered and the stream has a high $C\,V$, however, the rate of curvature changes rapidly beyond that percentage of time for which the run-off just equals the draft, as may be seen in Fig. 11.

Values of annual deficiency, to be subtracted from seasonal storage in order to produce the monthly storage curves, are taken from Fig. 12. Consider, for example, a stream the $C\,V$ of which is 0.3. The run-off at 95% of

the time is 0.565; the deficiency in respect to 1.0 draft is, therefore, 0.435, and, in respect to 0.9 draft, 0.335, etc. The seasonal storage for 0.0 constant, 95% of the time, 1.0 draft (Fig. 6) is 0.66, and subtracting the 0.435 annual deficiency from this leaves 0.225 as the 95% monthly storage value.

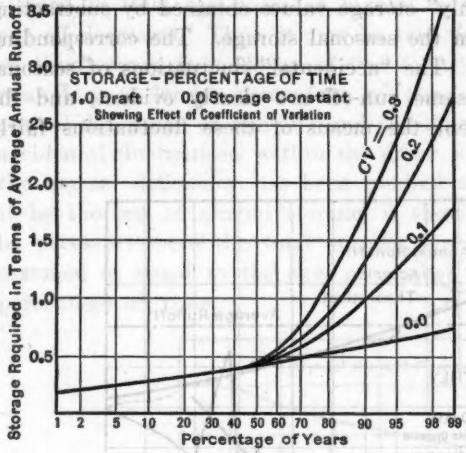


FIG. 11.

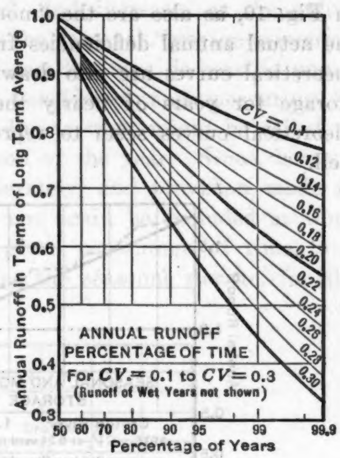


FIG. 12.

The annual storage for $C\,V = 0.3$ and 1.0 draft (Fig. 13) is 7.4 units of $C\,V$, or $7.4 \times 0.3 = 2.22$; and the total storage equals $2.22 + 0.225$, or 2.445 times the average annual run-off of the stream considered.

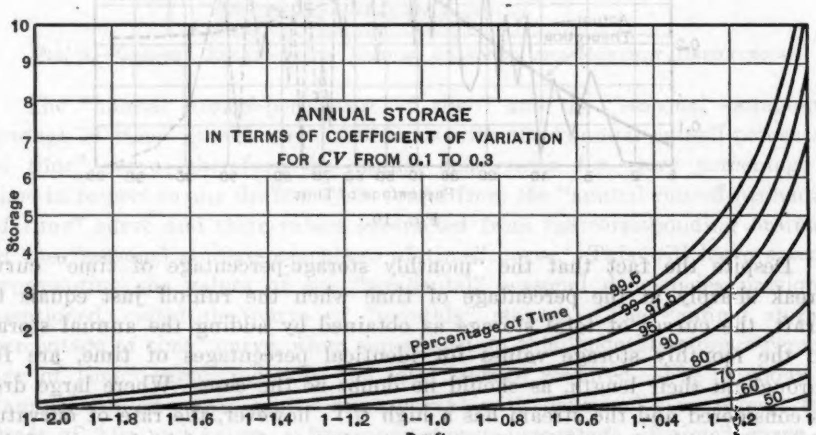


FIG. 13.

The complete storage curves as computed for 90%, 95%, and 99% of the time, are shown on Figs. 14, 15, and 16.

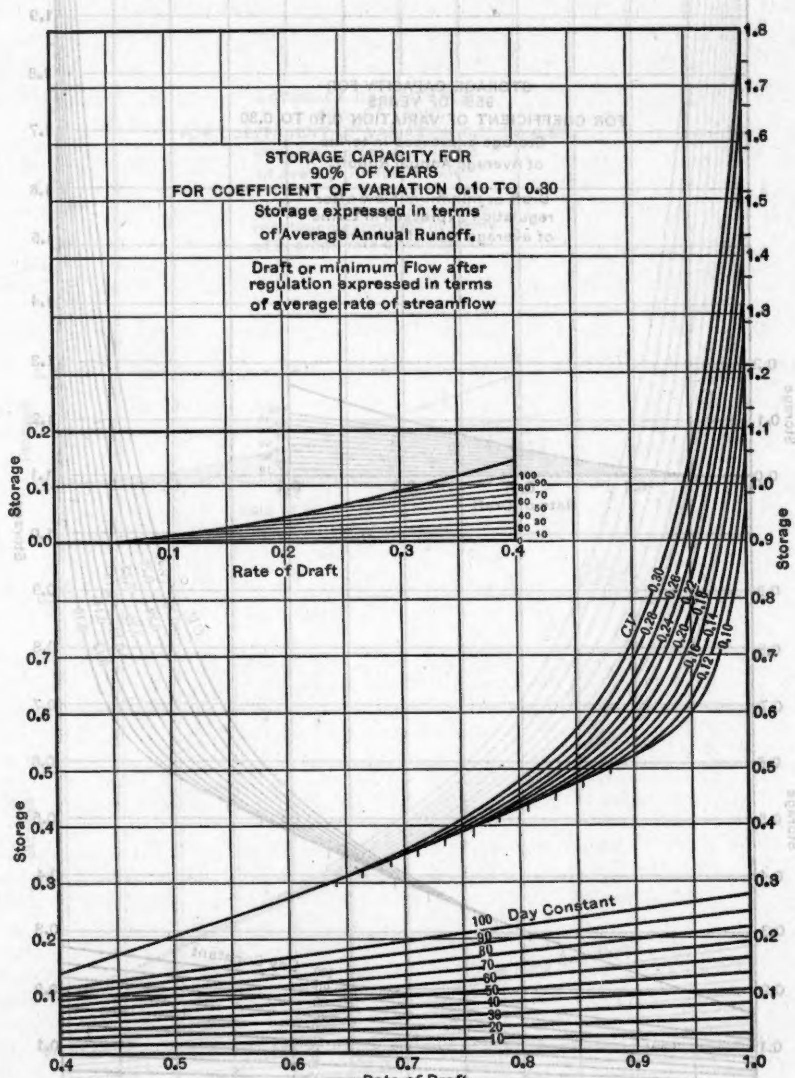

STORAGE CAPACITY FOR
90% OF YEARS
FOR COEFFICIENT OF VARIATION 0.10 TO 0.30
Storage expressed in terms
of Average Annual Runoff.
Draft or minimum Flow after
regulation expressed in terms
of average rate of streamflow
Storage
Rate of Draft
CV
Day Constant

FIG. 14.

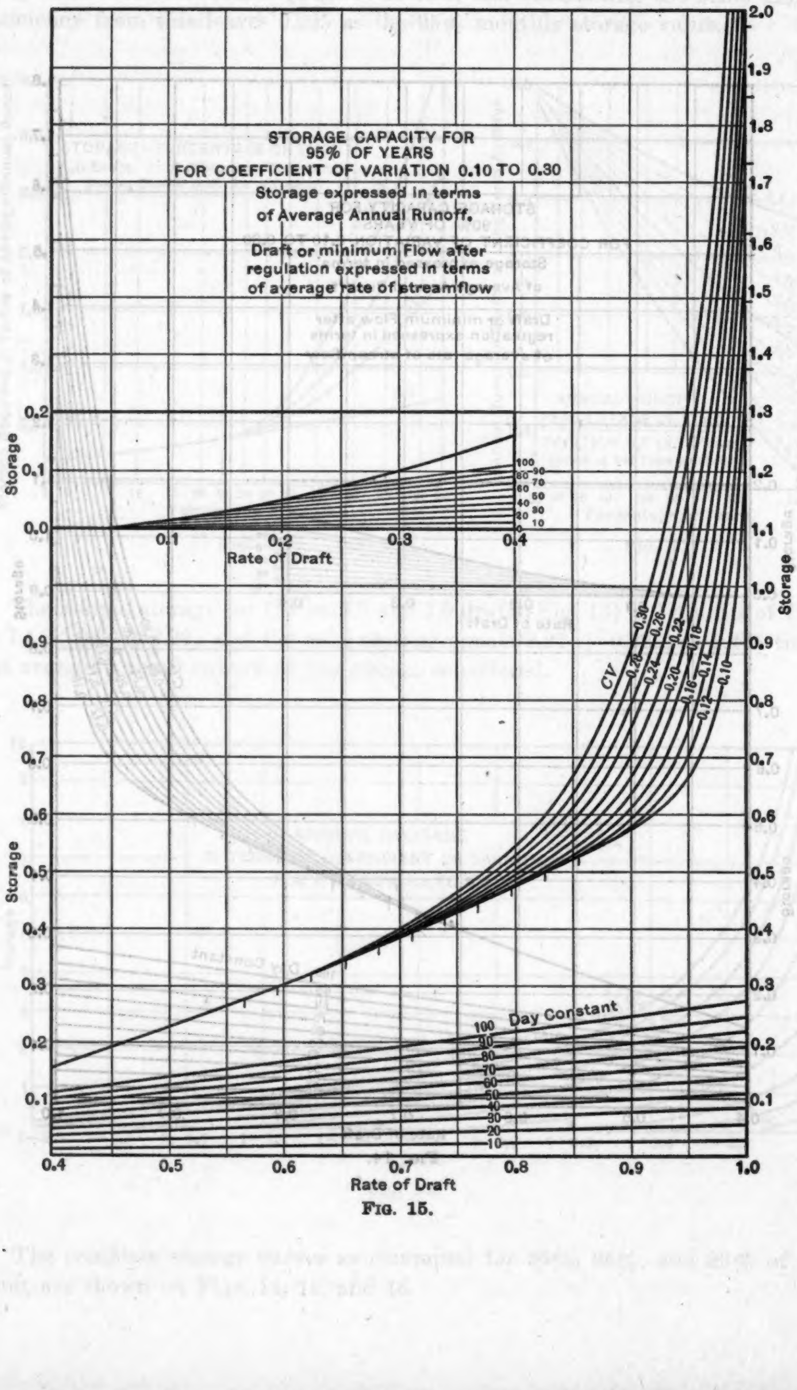
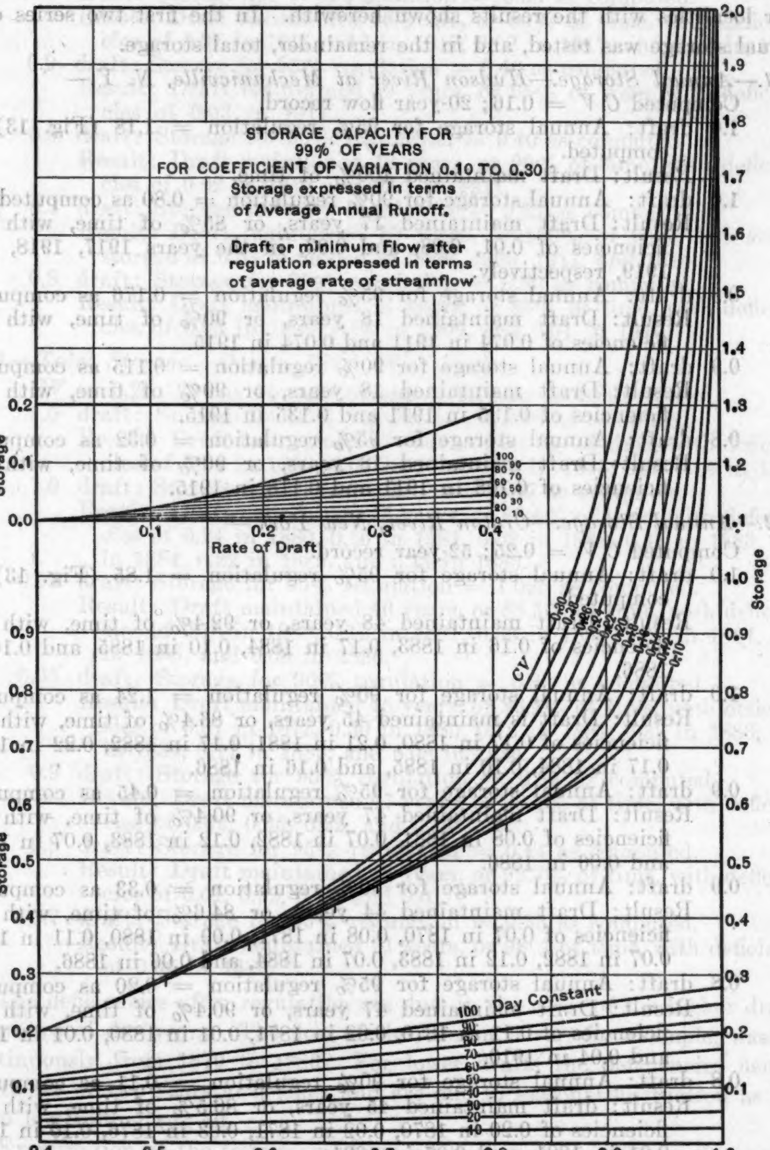

STORAGE CAPACITY FOR
95% OF YEARS
FOR COEFFICIENT OF VARIATION 0.10 TO 0.30
Storage expressed in terms
of Average Annual Runoff.
Draft or minimum Flow after
regulation expressed in terms
of average rate of stream flow
Storage
Rate of Draft
CV
0.30
0.28
0.26
0.24
0.22
0.20
0.18
0.16
0.14
0.12
0.10
Day Constant
100
90
80
70
60
50
40
30
20
10
Storage
Rate of Draft
Storage

FIG. 15.



STORAGE CAPACITY FOR
99% OF YEARS
FOR COEFFICIENT OF VARIATION 0.10 TO 0.30
Storage expressed in terms
of Average Annual Runoff.
Draft or minimum Flow after
regulation expressed in terms
of average rate of streamflow
Storage
Rate of Draft
CV
Day Constant

Fig. 16.

Tests of the Storage Curves

To test the dependability of the curves, storage has been taken as indicated for the computed $C\,V$, storage constant, percentage of time, and draft, for four localities with the results shown herewith. In the first two series only annual storage was tested, and in the remainder, total storage.

1.—Annual Storage.—Hudson River at Mechanicville, N. Y.—

Computed $C\,V = 0.16$; 20-year flow record.

1.0 draft: Annual storage for 95% regulation = 1.18 (Fig. 13) as computed.
Result: Draft maintained 100% of time.

1.0 draft: Annual storage for 90% regulation = 0.80 as computed.
Result: Draft maintained 17 years, or 85% of time, with deficiencies of 0.01, 0.08, and 0.03, in the years 1917, 1918, and 1919, respectively.

0.9 draft: Annual storage for 95% regulation = 0.176 as computed.
Result: Draft maintained 18 years, or 90% of time, with deficiencies of 0.074 in 1911 and 0.074 in 1915.

0.9 draft: Annual storage for 90% regulation = 0.115 as computed.
Result: Draft maintained 18 years, or 90% of time, with deficiencies of 0.135 in 1911 and 0.135 in 1915.

0.8 draft: Annual storage for 95% regulation = 0.32 as computed.
Result: Draft maintained 18 years, or 90% of time, with deficiencies of 0.108 in 1911 and 0.118 in 1915.

2.—Annual Storage.—Croton River, New York.—

Computed $C\,V = 0.25$; 52-year record.

1.0 draft: Annual storage for 95% regulation = 1.85 (Fig. 13) as computed.
Result: Draft maintained 48 years, or 92.4% of time, with deficiencies of 0.16 in 1883, 0.17 in 1884, 0.10 in 1885, and 0.16 in 1886.

1.0 draft: Annual storage for 90% regulation = 1.24 as computed.
Result: Draft is maintained 45 years, or 86.4% of time, with deficiencies of 0.17 in 1880, 0.21 in 1881, 0.17 in 1882, 0.22 in 1883, 0.17 in 1884, 0.10 in 1885, and 0.16 in 1886.

0.9 draft: Annual storage for 95% regulation = 0.45 as computed.
Result: Draft maintained 47 years, or 90.4% of time, with deficiencies of 0.08 in 1881, 0.07 in 1882, 0.12 in 1883, 0.07 in 1884, and 0.06 in 1886.

0.9 draft: Annual storage for 90% regulation = 0.33 as computed.
Result: Draft maintained 44 years, or 84.6% of time, with deficiencies of 0.07 in 1870, 0.08 in 1871, 0.09 in 1880, 0.11 in 1881, 0.07 in 1882, 0.12 in 1883, 0.07 in 1884, and 0.06 in 1886.

0.8 draft: Annual storage for 95% regulation = 0.20 as computed.
Result: Draft maintained 47 years, or 90.4% of time, with deficiencies of 0.11 in 1870, 0.02 in 1871, 0.01 in 1880, 0.01 in 1881, and 0.04 in 1910.

0.8 draft: Annual storage for 90% regulation = 0.11 as computed.
Result: draft maintained 45 years, or 86.5% of time, with deficiencies of 0.20 in 1870, 0.02 in 1871, 0.03 in 1876, 0.10 in 1880, 0.01 in 1881, and 0.07 in 1891.

3.—Total Storage.—Hudson River at Mechanicville, N. Y.—

$C\,V = 0.16$; 60-day storage constant; 20-year weekly flow record.

1.0 draft: Storage for 95% regulation = 1.45 as computed.
Result: Draft maintained 20 years, or 100% of time. Storage of 1.33 actually needed.

0.95 draft: Storage for 95% regulation = 0.65 as computed.
Result: Draft maintained 18 years, or 90% of time, with deficiencies of 0.02 and 0.07 in 1918 and 1919, respectively.
0.95 draft: Storage for 90% regulation = 0.53 as computed.
Result: Draft maintained 16 years, or 80% of time, with deficiencies of 0.01 in 1911, 0.12 in 1912, 0.02 in 1917, and 0.07 in 1919.
0.9 draft: Storage for 95% regulation = 0.46 as computed.
Result: Draft maintained 19 years, or 95% of time, with deficiencies of 0.03 in 1911.
0.9 draft: Storage for 90% regulation = 0.40 as computed.
Result: Draft maintained 18 years, or 90% of time, with deficiencies of 0.02 in 1910 and 0.07 in 1911.
0.8 draft: Storage for 95% regulation = 0.35 as computed.
Result: Draft maintained 20 years, or 100% of time, but actual storage of 0.33 is needed in 1908, 1909, 1911, and 1914.
0.8 draft: Storage for 90% regulation = 0.30 as computed.
Result: Draft maintained 17 years, or 85% of time, with deficiencies of 0.03 in 1908, 0.03 in 1909, and 0.01 in 1910.

4.—Total Storage.—Croton River, New York.—
CV = 0.25; 33-day storage constant; 52-year monthly record.
1.0 draft: Storage for 95% regulation = 2.06 as computed.
Result: Draft maintained 47 years, or 90.4% of time, with deficiencies of 0.24 in 1883, 0.07 in 1884, 0.22 in 1885, and 0.14 in 1886.
1.0 draft: Storage for 90% regulation = 1.47 as computed.
Result: Draft maintained 45 years, or 86.4% of time, with deficiencies of 0.24 in 1880, 0.16 in 1881, 0.02 in 1882, 0.41 in 1883, 0.07 in 1884, 0.22 in 1885, and 0.14 in 1886.
0.95 draft: Storage for 95% regulation = 1.025 as computed.
Result: Draft maintained 46 years, or 88.5% of time, with deficiencies of 0.16 in 1880, 0.12 in 1881, 0.33 in 1883, 0.02 in 1884, 0.17 in 1885, and 0.08 in 1886.
0.95 draft: Storage for 90% regulation = 0.82 as computed.
Result: Draft maintained 45 years, or 86.4% of time, with deficiencies of 0.07 in 1872, 0.30 in 1880, 0.12 in 1881, 0.33 in 1883, 0.02 in 1884, 0.17 in 1885, and 0.08 in 1886.
0.9 draft: Storage for 95% regulation = 0.69 as computed.
Result: Draft maintained 47 years, or 90.4% of time, with deficiencies of 0.02, 0.05, 0.22, and 0.11.
0.8 draft: Storage for 95% regulation = 0.44 as computed.
Result: Draft maintained 48 years, or 92.3% of time, with deficiencies of 0.01, 0.02, 0.11, and 0.01.
0.7 draft: Storage for 95% regulation = 0.33 as computed.
Result: Draft maintained 51 years, or 98% of time, with deficiency of 0.06 in 1880.

The deficiencies after regulation are due, in the cases of the higher drafts, to groups of dry years. The flow of the Croton River, for example, was low continuously from 1879 to 1886. For lower drafts the deficiencies usually occur one or two years at a time, and are due to seasonal fluctuation as well as to low annual run-off.

Examination of the tests seems to indicate that even a 20-year flow record is not sufficient in itself to provide consistent, reasonable results when various drafts and percentages of time are considered. Thus, the theoretical annual storage for 95% regulation of the Hudson River for 1.0 draft maintains the flow 100% of the time, while the 90% storage maintains it only 85% of the

time. The annual storage for 90% regulation, 0.9 draft, is also sufficient for 95% of time. There seems to be no assurance that the grouping of dry years in one 20-year record will be similar, in respect to storage requirements, to that of the next 20 years.

The storage curves, Figs. 14, 15, and 16, are designed as suitable for streams having CV of 0.30 or less. For larger coefficients the annual storage computed from Fig. 5 will be too small, since the coefficient of skew should be greater, corresponding to more years of deficient run-off. However, a test has been made of the annual storage required for the Murray River, at Mildura, Australia, having a computed CV of 0.53, as based on a run-off of 48 calendar years. The theoretical annual storage needed, as taken from Fig. 5, is as follows:

1.0 draft......95% annual storage = 7.39 CV = 3.92 average run-off
" " ......90% " " = 4.97 CV = 2.64 " "
" " ......80% " " = 2.80 CV = 1.48 " "

Using these computed storages it was found that the theoretical 95% storage maintained the draft for 92% of the years; the 90% storage, for 85% of the years; and the 80% storage, for 75% of the years.

The annual run-off of this river frequently reaches 175 to 200% of its average, or falls as low as 55% of its average, while in 1903 the run-off fell to 0.16 of its average. The computed coefficient of skew (CS) is 1.48, whereas the storage curves used were based on a CS of 0.6.

Percentage of Years in Which Partial Depletion Occurs

In Fig. 17 an "annual run-off-percentage of time" curve is drawn. Assuming no regulation by the use of storage, a nominal draft or installed wheel capacity equivalent to 0.947 mean stream flow could operate at full capacity for 60% of the time, after which the rate of flow becomes deficient, dropping to 0.0 at 100% of the time, as shown. Computing the area below the draft line and the run-off curve it is found that with installed capacity of 0.947, an average use of 0.907 could be obtained, and the natural flow would exceed this average during 72% of the time. The deficiencies in respect to this average use are measured by the area, *b*, which is equal to the area of excesses, *a*.

Computing thus the average use available for various installations, or "nominal drafts", a curve may be drawn as in Fig. 18. In this diagram the nominal draft curve is a "run-off-percentage of time" curve, and drafts are expressed in terms of CV. Reading from this diagram, for example, a draft of $1 - 0.33\,CV$ may be maintained 60% of the time, and the average use available to an installation equivalent to this draft, is $1 - 0.57CV$, found at the intersection of the lower curve with the 60% ordinate. Following this average use of 1 — 0.57 horizontally to the right, its intersection with the run-off or nominal draft curve occurs at 70% of the time, indicating that this average use may be exceeded during 70% of the years. If the stream being considered has $CV = 0.16$, the nominal draft of $1 - 0.33\ CV = 1 - (0.33 \times 0.16) = 0.947$, and the average use = 0.907, as in Fig. 17.

If, however, it is a minimum use of 0.907 that is required, water must be stored, equal to the deficiencies measured by the area, *b*, Fig. 17, and this must be taken from flow that exceeds the nominal draft or installed capacity. It is reasoned that as a general condition this storage probably will be taken about as represented by the area, *a*, which equals *b*. If this is true, then with complete or 100% regulation, Fig. 12 would show that with a draft or capacity of 0.907, there will be 60 years during which the run-off is so high that the reservoir is completely filled and waste flow occurs; 10 years (between 60 and 70% of the time) during which the run-off exceeds the draft, but not sufficiently to make good the entire depletion; and 30 years (between 70 and 100%) during which the run-off is deficient in respect to the draft, and storage must be drawn upon.

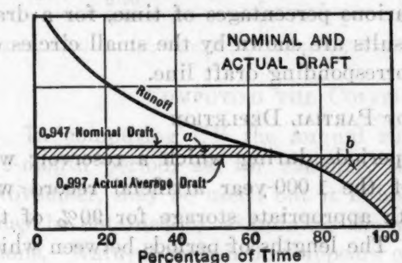


FIG. 17.

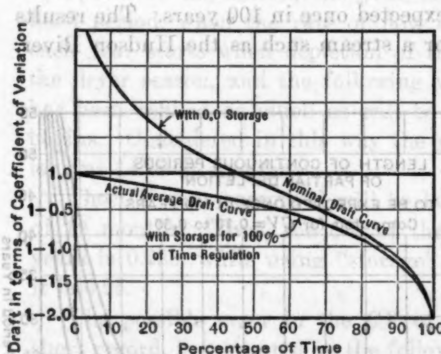


FIG. 18.—PERCENTAGE OF YEARS RESERVOIR WILL BE COMPLETELY FULL.

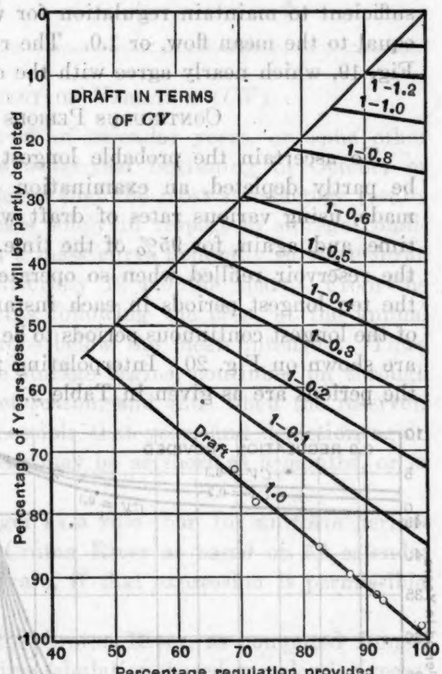


FIG. 19.—PERCENTAGE OF YEARS PARTIAL DEPLETION OCCURS.

Following this argument the two curves of Fig. 18 represent conditions with no annual storage and with storage for 100% of the time. Thus, for 1.0 draft (equal to the average stream flow) with complete regulation, the reservoir will be exactly full 1 year in 100, or 1 000, or whatever period is assumed as 100% of the time, and partial depletion will continue for the remainder of the time. With no storage, the reservoir (of 0.0 capacity) will be full during the 46 wet years in each 100, and partly depleted during the remaining 54 years. The intersection with the two curves of a horizontal line representing any draft shows the percentage of time the reservoir will be full with complete storage and with no storage.

For intermediate conditions, as, for example, when regulation for 90% of the time is provided, Fig. 19 indicates the relations. This diagram is formed by replotting the values from the curves of Fig. 18, and connecting them by straight lines. As an example of its use assume a stream having a $CV = 0.2$, and a draft equal to 0.9 of the mean stream flow or $1 - 0.5\ CV$. The diagram shows that this draft would be maintained 67% of the years without any storage. Now, if storage is provided to regulate for 90% of the time, the reservoir will be in a partly depleted condition during 42% of the years.

Although perhaps Fig. 19 is not entirely conclusive, it does take into account the most severe condition of draft. As a check on its construction an examination of the 1 000-year artificial flow record was made with storages sufficient to maintain regulation for various percentages of time, for a draft equal to the mean flow, or 1.0. The results are shown by the small circles on Fig. 19, which nearly agree with the corresponding draft line.

Continuous Periods of Partial Depletion

To ascertain the probable longest periods during which a reservoir will be partly depleted, an examination of the 1 000-year artificial record was made, using various rates of draft with appropriate storage for 90% of the time, and, again, for 95% of the time. The lengths of periods between which the reservoir refilled when so operated, were tabulated, and the averages of the ten longest periods in each instance were taken as the probable lengths of the longest continuous periods to be expected once in 100 years. The results are shown on Fig. 20. Interpolating for a stream such as the Hudson River, the periods are as given in Table 2.

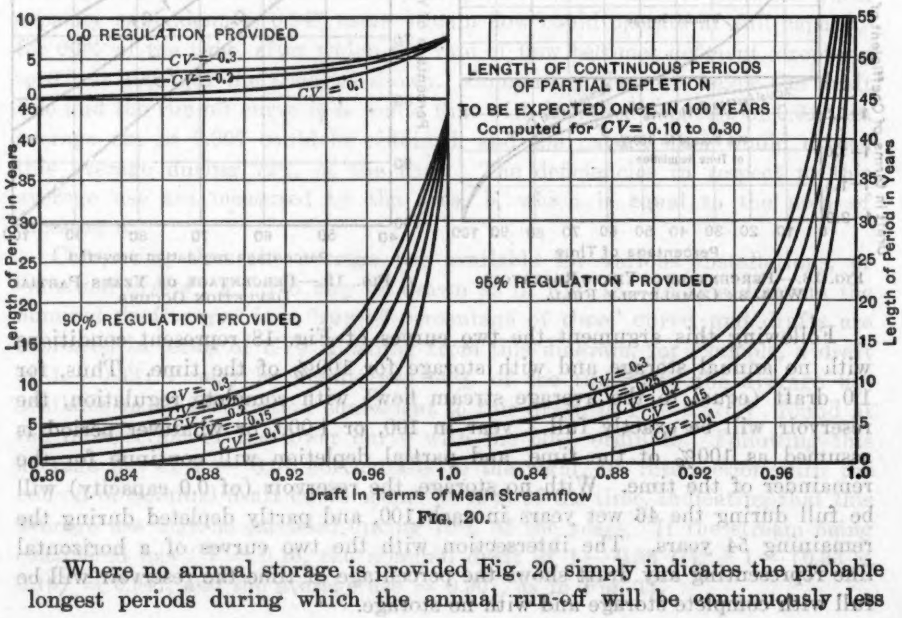


Fig. 20.

Where no annual storage is provided Fig. 20 simply indicates the probable longest periods during which the annual run-off will be continuously less

than an assumed draft. For the Hudson, during a period of about 5 years once in 100 years, the run-off will be continuously less than 0.98; and for a 3-year period, it will be less than 0.90.

TABLE 2.—Continuous Periods of Partial Depletion for Hudson River ($CV = 0.16$).

Assumed draft, (mean stream flow).	Percentage of regulation.	Length of depletion period, in years.
0.98	90	21
0.98	95	27
0.95	90	10
0.95	95	12
0.90	90	5
0.90	95	6

Computing the Coefficient of Variation (CV)

In computing CV the annual run-off of calendar years, or some other 12-month period, as, for example, the water-year beginning in October or November, may be used; but it is believed that the most satisfactory results may be obtained by examining the years solely in respect to storage conditions. Natural stream flow appears on a mass curve generally in wave form, with crests approximately but not uniformly 12 months apart. From the viewpoint of regulation, the year begins following the crest at the annual flood period, when the rate of flow drops below the mean intensity. Thus, each year starts when depletion of the storage begins, continues on through the dryer season, and the following wet season, and ends when the reservoir has been refilled as much as will be possible that year, and depletion again begins. Considered in this way the year may be as short as 9 months, or as long as 14 or 15 months.

The CV thus computed will be larger as a rule than for uniform periods of 12 months. Thus, the CV of the Croton River as based on 52 calendar years is 0.238, while using "storage" years, if that expression is permissible, it is 0.25.

The possible error in the CV of the Croton River, as computed from a short record, is suggested by the following tabulation, based on calendar years:

52-year period.....CV .... = 0.238
20-year periods....CV ranges from 0.226 to 0.276; ave. = 0.251
10-year periods....CV " " 0.157 " 0.320; " = 0.243
5-year periods....CV " " 0.049 " 0.405; " = 0.245

Storage on a Tributary

Not infrequently a reservoir is located some distance above the point of use, or on a branch stream, providing a condition wherein the average run-off at the power site or point of use considerably exceeds the average run-off tributary to the reservoir. In such instances, as part of the stream flow that reaches the point of use does not pass through the reservoir, a question arises as to the usefulness of the storage.

As the position of the reservoir does not affect the natural flow at any point, the amount of storage which must be released during dry seasons is identical for all reservoir locations; therefore, the limiting condition lies in the ability of the catchment area tributary to the reservoir to supply the storage which must be released.

For the purpose of this discussion the whole run-off or stream flow passing the point of use will be assumed as unity; the portion that is tributary to the reservoir and therefore may be controlled, will be called the "tributary" run-off, and its proportion to the whole will be called the "tributary ratio"; and the portion passing the point of use without going through the reservoir will be called "main-stream" portion, and its proportion to the whole will be called the "main-stream ratio". In Fig. 21, the run-off at *A*, the point of use, corresponds to a catchment area of 100 sq. miles, of which 60 sq. miles is tributary to the reservoir, *B*. The "tributary ratio" is, therefore, $\frac{60}{100}$, or 0.6, while the "main-stream ratio" is $\frac{40}{100}$, or 0.4.

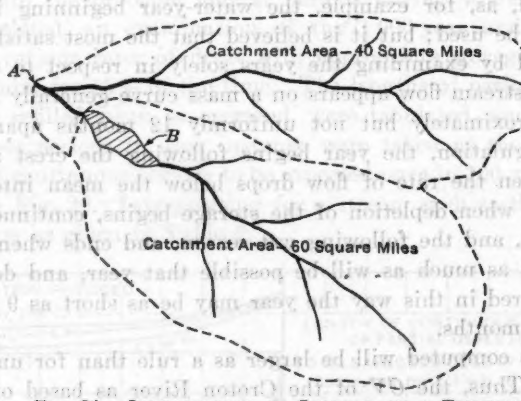


FIG. 21.—ILLUSTRATION OF STORAGE ON A TRIBUTARY.

Flow that passes into the reservoir may be controlled; but when this flow is completely shut off while the flow of the main stream passes the point of use at rates exceeding the assumed draft, the excess is lost to use. On this excess depends the usefulness of the storage, so an effort will be made to find its value.

A "daily run-off-percentage of time" curve embracing a considerable period of years may be assumed to give a good approximation of the relations between various rates of flow and the periods during which such flow is exceeded. As the area enclosed below such a curve measures the average run-off, the areas above lines representing various rates of flow or draft measure the average quantities of excess, referred to these drafts. From study of a number of "daily run-off-percentage of time" curves of Eastern streams for which the storage constants had been computed, a set of theoretical curves were drawn (Fig. 22). Presumably these curves would not fit all streams having

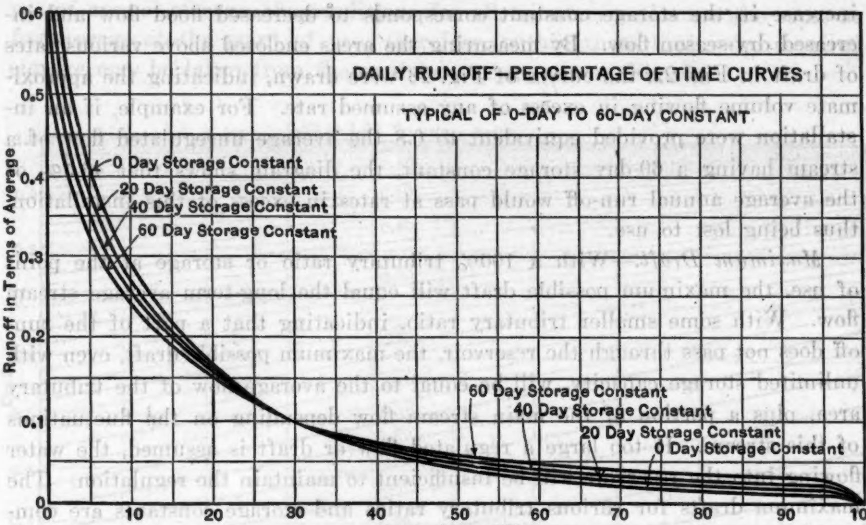

DAILY RUNOFF—PERCENTAGE OF TIME CURVES
TYPICAL OF 0-DAY TO 60-DAY CONSTANT
Runoff in Terms of Average
0.6
0.5
0.4
0.3
0.2
0.1
0
0 Day Storage Constant
20 Day Storage Constant
40 Day Storage Constant
60 Day Storage Constant
60 Day Storage Constant
40 Day Storage Constant
20 Day Storage Constant
0 Day Storage Constant
0
10
20
30
40
50
60
70
80
90
100
Percentage of Time

FIG. 22.

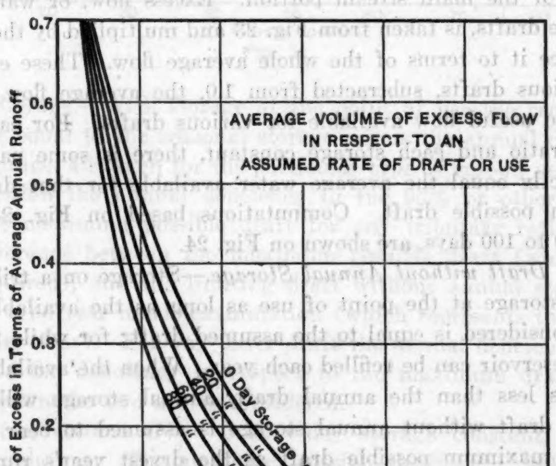

AVERAGE VOLUME OF EXCESS FLOW
IN RESPECT TO AN
ASSUMED RATE OF DRAFT OR USE
Volume of Excess in Terms of Average Annual Runoff
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0 Day Storage Constant
20
40
60
80
0
1.0
2.0
3.0
4.0
5.0
6.0
Draft or Use in Terms of Average Flow

FIG. 23.

similar storage constants, but would approximate them and show that an increase in the storage constant corresponds to decreased flood flow and increased dry-season flow. By measuring the areas enclosed above various rates of draft in Fig. 22, the curves of Fig. 23 were drawn, indicating the approximate volume flowing in excess of any assumed rate. For example, if an installation were provided equivalent to 0.8 the average unregulated flow of a stream having a 60-day storage constant, the diagram shows that 42.5% of the average annual run-off would pass at rates in excess of this installation, thus being lost to use.

Maximum Draft.—With a 100% tributary ratio or storage at the point of use, the maximum possible draft will equal the long-term average stream flow. With some smaller tributary ratio, indicating that a part of the run-off does not pass through the reservoir, the maximum possible draft, even with unlimited storage capacity, will be equal to the average flow of the tributary area, plus a portion of the main stream flow depending on the fluctuations of this stream. If too large a regulated flow or draft is assumed, the water flowing into the reservoir will be insufficient to maintain the regulation. The maximum drafts for various tributary ratios and storage constants are computed from Fig. 23, as follows: Various drafts at the point of use are assumed, expressed in terms of the average stream flow of the whole area. Dividing these drafts by the main-stream ratio reduces them to terms of average flow of the main stream portion. Excess flow, or water lost in relation to these drafts, is taken from Fig. 23 and multiplied by the main-stream ratio to reduce it to terms of the whole average flow. These excesses in relation to various drafts, subtracted from 1.0, the average flow of the whole area, show the useful flow available for various drafts. For each proportion of tributary ratio and each storage constant, there is some particular draft that will exactly equal the average water available for that draft. This is the maximum possible draft. Computations based on Fig. 23 for storage constants of 0 to 100 days, are shown on Fig. 24.

Maximum Draft without Annual Storage.—Storage on a tributary is just as useful as storage at the point of use as long as the available flow in the dryest year considered is equal to the assumed draft; for while this condition obtains, the reservoir can be refilled each year. When the available flow of the dryest year is less than the annual draft, annual storage will be required. The limiting draft without annual storage is assumed to bear the same relation to the maximum possible draft as the dryest year's run-off bears to the average year's run-off. This assumption is made on the theory that the characteristic "run-off-percentage of time" curve for any dry year is of the same type as that of the average year, as drawn in Fig. 22; every ordinate of the dry-year curve being reduced however, in the proportion indicated by the CV of the stream considered. If this is true, in the dry year both the draft and the water available to that draft are reduced in the same proportion, and as all the available water is used in the average year, so also will it all be used in the dry year.

Seasonal Storage.—As long as the assumed draft does not necessitate the use of annual storage, the conditions for all tributary ratios are the same as for storage at the point of use; therefore, within these limits the required storage may be taken from the seasonal storage curves, Fig. 6.

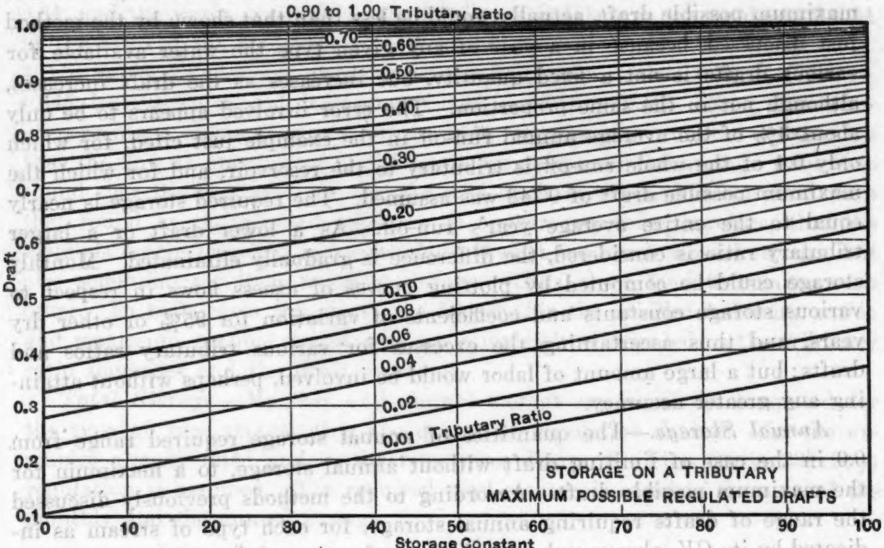


FIG. 24.

Monthly Storage.—With storage at the point of use the required total is assumed to be equal to the seasonal storage minus the annual deficiency plus the annual storage, all taken for the particular type of run-off year considered. In this discussion the annual deficiency in the 95% or other dry year, in respect to the maximum possible draft for any tributary ratio, is assumed to be the difference between the maximum possible draft (which represents the water required), and the limiting draft without annual storage, for the 95% or other type year under examination (which represents the water available in this dry year). For any lower draft the annual deficiency is assumed to be equal to the deficiency in respect to the maximum draft, minus the difference between the two drafts considered.

For example, assume $CV = 0.10$, 0-day storage constant, 0.4 tributary ratio, 95% regulation: The maximum possible draft (Fig. 24) is 0.748; the limiting draft without annual storage is $0.748 \times 0.855^* = 0.640$. The 95% year annual deficiency is $0.748 - 0.640 = 0.108$. For any lower draft, as 0.69, for example, the annual deficiency is assumed as 0.108 minus the difference between the two drafts, or, Draft $0.748 -$ Draft $0.690 = 0.058$; and $0.108 - 0.058 = 0.050$ is the annual deficiency for Draft 0.69. The seasonal storage (Fig. 6) for the 0.748 draft $= 0.428$, and subtracting from this the 0.108 annual deficiency, the remainder or monthly storage is found to be

* This is the 95% run-off for $CV = 0.10$ (Fig. 12).

0.320. The monthly storage for 0.69 and other drafts would be computed in the same manner.

This method is not entirely consistent with the assumption that the flow for every percentage of time for a dry year was a fixed fraction of that for an average year. The annual deficiency for some dry year, in respect to the maximum possible draft, actually would be less than that shown by the method just discussed, because in a year of any given type the water available for various drafts is not a fixed quantity, but increases as the draft increases, although not in the same proportion. The error involved appears to be only about 3% of the average annual run-off in the example just cited, for which only 0.4 of the whole run-off is tributary to the reservoir, and for which the maximum possible draft of 0.748 was assumed. The required storage is nearly equal to the entire average year's run-off. As a lower draft or a larger tributary ratio is considered, the difference is gradually eliminated. Monthly storage could be computed by plotting curves of excess flows in respect to various storage constants and coefficients of variation for 95% or other dry years, and thus ascertaining the excesses for various tributary ratios and drafts; but a large amount of labor would be involved, perhaps without attaining any greater accuracy.

Annual Storage.—The quantities of annual storage required range from 0.0 in the case of limiting draft without annual storage, to a maximum for the maximum possible draft. According to the methods previously discussed the range of drafts requiring annual storage, for each type of stream as indicated by its CV, always embraces the same fraction of the maximum possible draft. For example, again assuming 95% regulation, $CV = 0.10$, 0-day constant: The 95% year run-off (Fig. 12) is 0.855; the maximum possible draft for a 0.4 tributary ratio (Fig. 24) is 0.748; and the limiting draft without annual storage is $0.748 \times 0.855 = 0.640$. Now, for a 0.1 tributary ratio the maximum possible draft (Fig. 24) is 0.365, and, therefore, the limiting draft without annual storage is $0.365 \times 0.855 = 0.312$.

It is now assumed that the annual storage in the case of storage at the point of use, or 100% tributary ratio, depends on the relative drafts considered, so that if the relative drafts are expressed in terms of maximum possible draft the storage for any tributary ratio can be found as the product of the annual storage for the given draft in terms of the maximum possible draft, as required for a 100% tributary ratio, by the maximum possible draft for the given tributary ratio. For example, with $CV = 0.10$ and 0-day storage constant: The maximum draft for a 100% tributary ratio (Fig. 24) is 1.0 and requires 95% year annual storage (Fig. 13) of 0.739. For a 0.9 draft an annual storage of 0.05 is required. Now, with a 0.4 tributary ratio, the maximum possible draft is, as noted, 0.748 and since this represents 100% of the average available run-off the required annual storage is computed as $0.748 \times 0.739 = 0.553$; and a draft of 0.9 maximum, or $0.9 \times 0.748 = 0.673$, requires annual storage of $0.748 \times 0.05 = 0.037$.

If the assumption is true—and it appears not unreasonable—that the flow for every typical year follows the same characteristics as those indicated by

the long-term "run-off-percentage of time" curve, then, in any dry year, considering the main stream flow in respect to the maximum possible draft, actually there will be more water available than this method assumes, as was explained under "Monthly Storage."* For example, assuming a 0.5 tributary ratio, a 0-day constant, $CV = 0.16$: The maximum possible draft (Fig. 24) is 0.84, and the water available in the 95% year is taken as the product of 0.84 by the ratio of the 95% year run-off to the average year run-off, which (Fig. 12) is 0.768. The water available in the 95% year is then $0.84 \times 0.768 = 0.645$, leaving an annual deficiency of $0.84 - 0.645 = 0.195$. Now, if a "run-off-percentage of time" curve for the 95% year is drawn on the assumption that the flow at every point is 0.768 as large as that of the average year, it is found that the flow available for the 0.84 draft is 0.680, or 0.035 more than that found by the adopted method. It appears possible, therefore, that the methods herein proposed may provide somewhat too much annual storage, but also somewhat too little monthly storage. Thus, these tend to balance each other. In both cases the errors are largest for the smaller tributary ratios and maximum possible drafts, and are completely eliminated as the tributary ratio increases and the draft decreases.

Total Storage.—Monthly and annual storage are similarly computed for the type of year representing the degree of regulation desired. In the examples already cited and in the remaining diagrams (Figs. 25 to 33) 95% regulation is assumed. The required total storage is equal to the sum of the monthly and the annual storage.

Summary of Computations

1.—The maximum possible draft with unlimited storage (Fig. 24) is affected by the storage constant and the tributary ratio, and is determined from theoretical "run-off-percentage of time" curves (Fig. 22) which are computed to show the characteristic daily fluctuations typical of various storage constants.

The maximum possible draft is that draft in respect to which the average water available is just sufficient. It is not affected by the percentage of time the regulation is to be maintained.

2.—The limiting draft without annual storage, is affected by the storage constant, the CV of the annual flow, the tributary ratio, and the percentage of time that regulation is to be maintained.

The limiting draft is the product of the maximum possible draft by the ratio of the run-off of the dryest year considered to the run-off of the average year.

3.—The annual deficiency is affected by the storage constant, the CV, the tributary ratio, and the percentage of time that regulation is to be maintained.

For the maximum possible draft the annual deficiency equals the maximum possible draft (Paragraph 1) minus the limiting draft (Paragraph 2). For any lower draft it equals this quantity diminished by the difference between the maximum possible draft and the given draft.

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* See p. 1943.

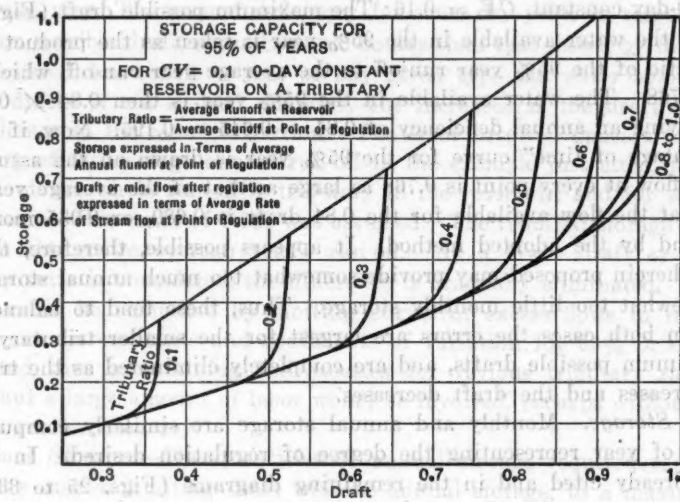
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV= 0.1 0-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0
Storage
Draft
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 25.

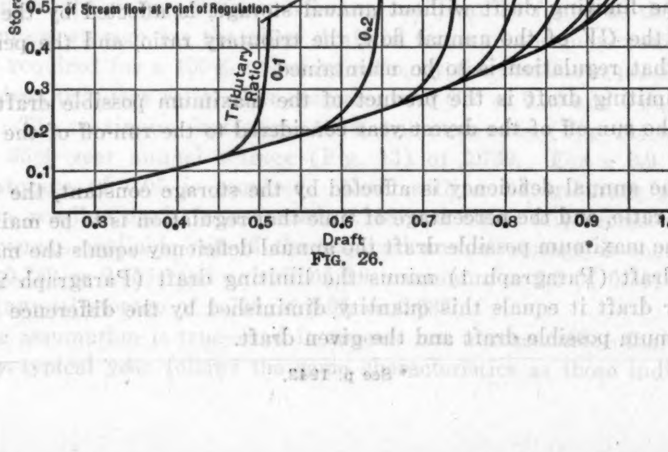
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV= 0.1 50-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0
Storage
Draft
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 26.

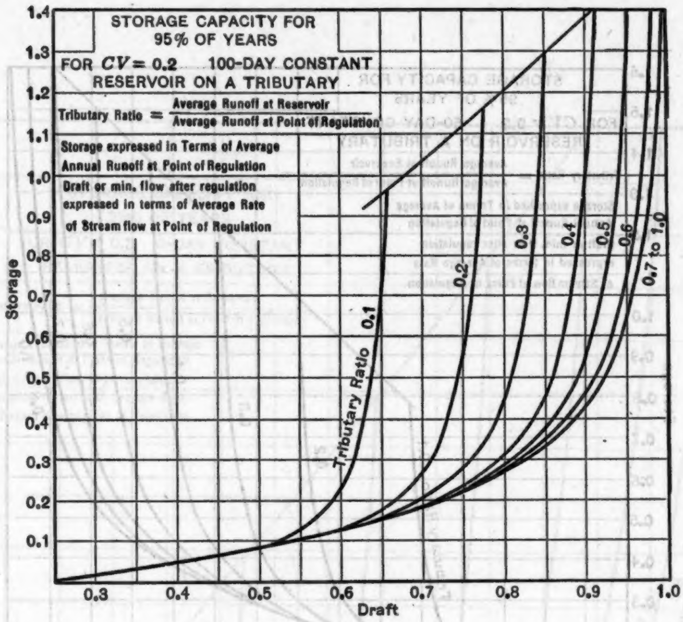
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV = 0.2 100-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Storage
Draft
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7 to 1.0

FIG. 27.

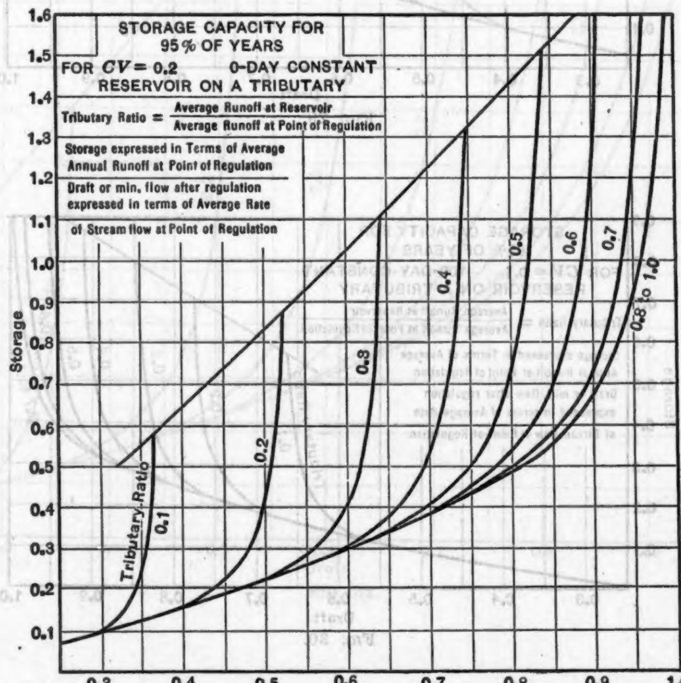
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV = 0.2 0-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Storage
Draft
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0

FIG. 28.

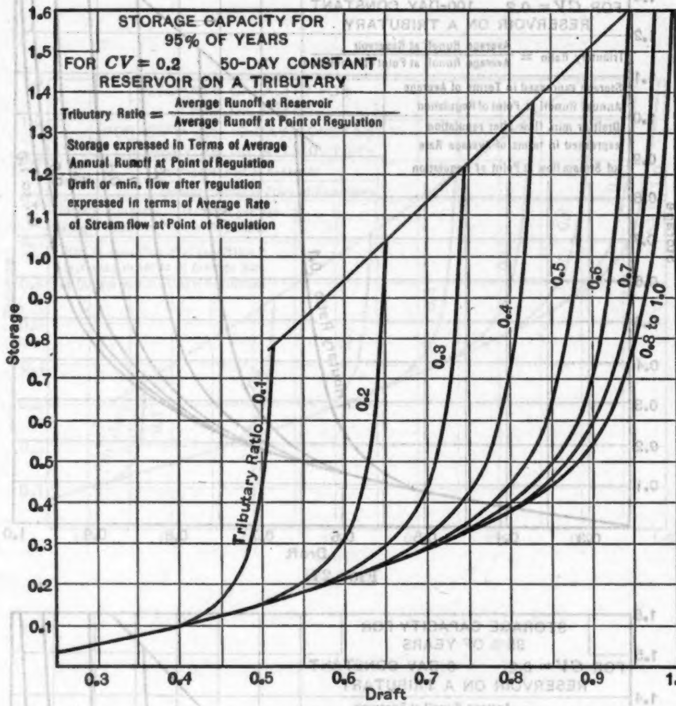
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV = 0.2 50-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average
Annual Runoff at Point of Regulation
Draft or min. flow after regulation
expressed in terms of Average Rate
of Stream flow at Point of Regulation
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0
Storage
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
1.6
Draft
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 29.

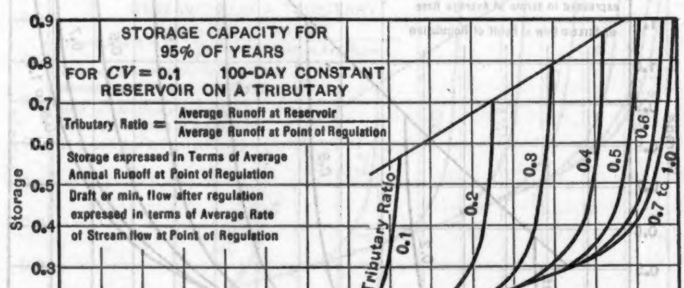
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV = 0.1 100-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average
Annual Runoff at Point of Regulation
Draft or min. flow after regulation
expressed in terms of Average Rate
of Stream flow at Point of Regulation
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7 to 1.0
Storage
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
Draft
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 30.

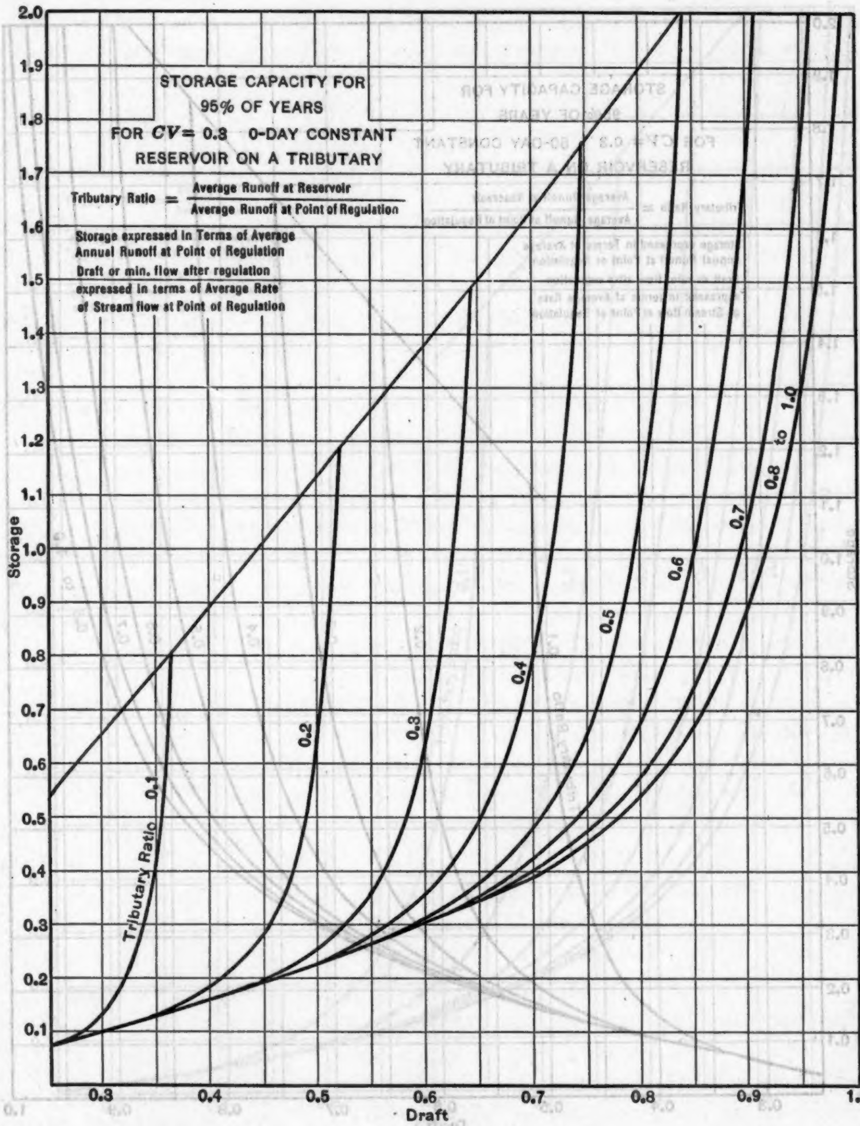
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV= 0.3 0-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average
Annual Runoff at Point of Regulation
Draft or min. flow after regulation
expressed in terms of Average Rate
of Stream flow at Point of Regulation
Tributary Ratio 0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0
Storage
Draft
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 31.

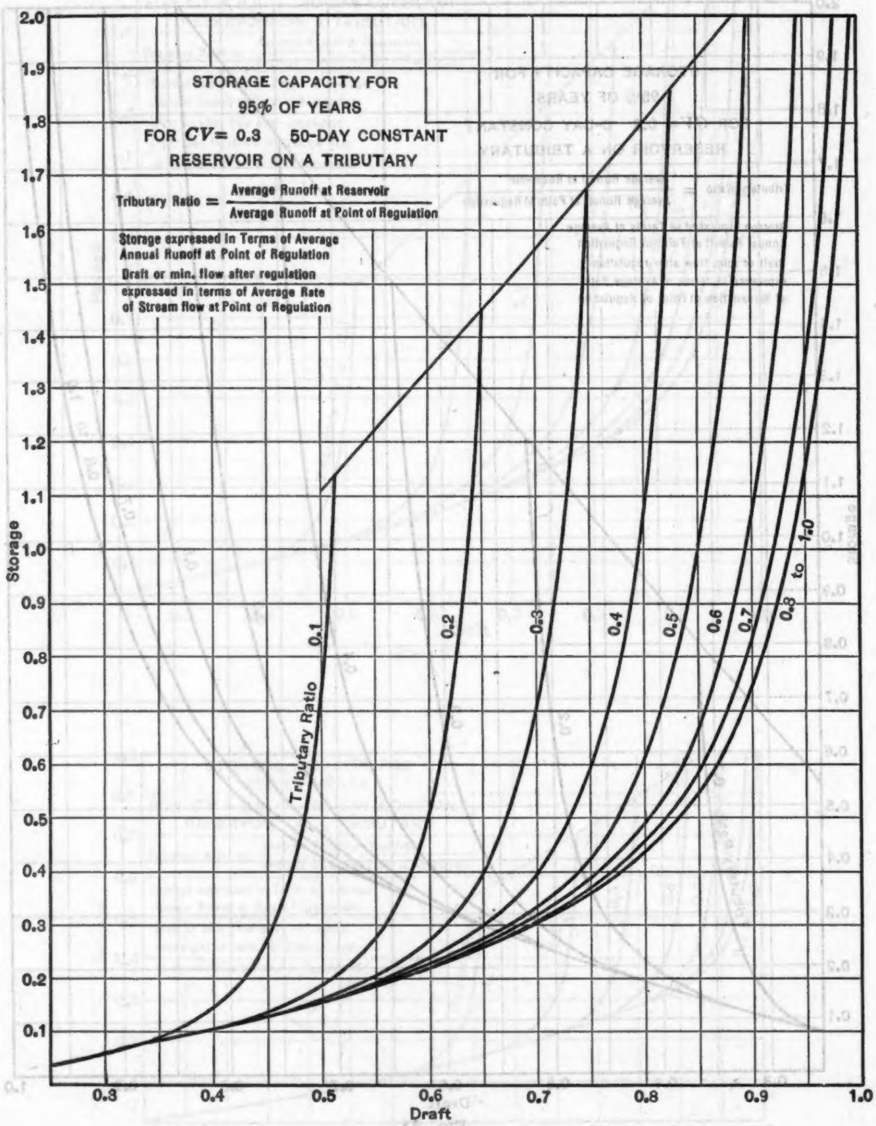
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV= 0.3 50-DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8 to 1.0
Storage
Draft

FIG. 32.

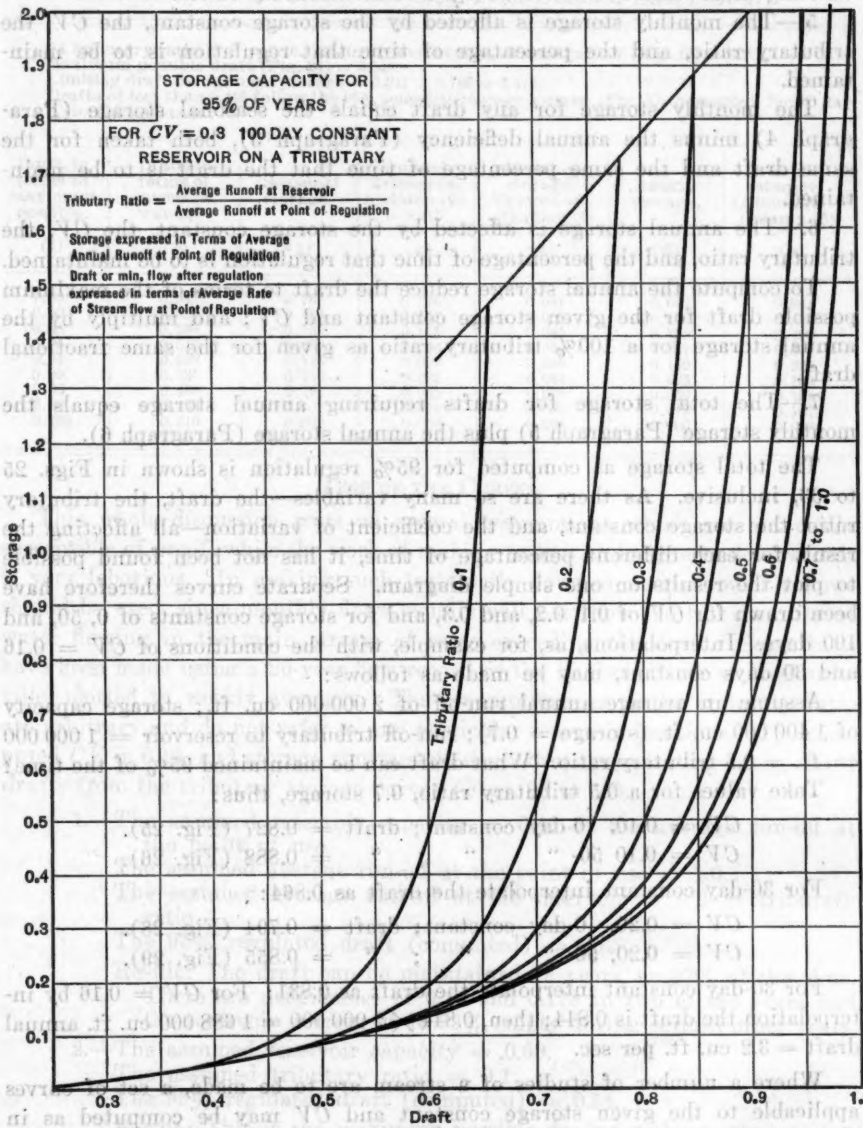
STORAGE CAPACITY FOR
95% OF YEARS
FOR CV = 0.3 100 DAY CONSTANT
RESERVOIR ON A TRIBUTARY
Tributary Ratio = Average Runoff at Reservoir / Average Runoff at Point of Regulation
Storage expressed in Terms of Average Annual Runoff at Point of Regulation
Draft or min. flow after regulation expressed in terms of Average Rate of Stream flow at Point of Regulation
Tributary Ratio
0.1
0.2
0.3
0.4
0.5
0.6
0.7 to 1.0
Storage
Draft
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2.0
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 33.

4.—The seasonal storage is affected by the storage constant and the percentage of time that regulation is to be maintained.

Seasonal storage (Figs. 6 and 7) is the total storage required below the limiting draft without annual storage, for all tributary ratios.

5.—The monthly storage is affected by the storage constant, the *CV*, the tributary ratio, and the percentage of time that regulation is to be maintained.

The monthly storage for any draft equals the seasonal storage (Paragraph 4) minus the annual deficiency (Paragraph 3), both taken for the same draft and the same percentage of time that the draft is to be maintained.

6.—The annual storage is affected by the storage constant, the *CV*, the tributary ratio, and the percentage of time that regulation is to be maintained.

To compute the annual storage reduce the draft to terms of the maximum possible draft for the given storage constant and *CV*; and multiply by the annual storage for a 100% tributary ratio as given for the same fractional draft.

7.—The total storage for drafts requiring annual storage equals the monthly storage (Paragraph 5) plus the annual storage (Paragraph 6).

The total storage as computed for 95% regulation is shown in Figs. 25 to 33, inclusive. As there are so many variables—the draft, the tributary ratio, the storage constant, and the coefficient of variation—all affecting the result for each different percentage of time, it has not been found possible to plot the results on one simple diagram. Separate curves therefore have been drawn for *CV* of 0.1, 0.2, and 0.3, and for storage constants of 0, 50, and 100 days. Interpolations, as, for example, with the conditions of $CV = 0.16$ and 30 days constant, may be made as follows:

Assume an average annual run-off of 2 000 000 cu. ft.; storage capacity of 1 400 000 cu. ft. (storage = 0.7); run-off tributary to reservoir = 1 000 000 cu. ft. = 0.5 tributary ratio. What draft can be maintained 95% of the time?

Take values for a 0.5 tributary ratio, 0.7 storage, thus:

$CV = 0.10$, 0-day constant; draft = 0.827 (Fig. 25).
$CV = 0.10$ 50- " " ; " = 0.888 (Fig. 26).

For 30-day constant interpolate the draft as 0.864:

$CV = 0.20$, 0-day constant; draft = 0.794 (Fig. 28).
$CV = 0.20$, 50- " " ; " = 0.855 (Fig. 29).

For 30-day constant interpolate the draft as 0.831: For $CV = 0.16$ by interpolation the draft is 0.844; then, $0.844 \times 2\,000\,000 = 1\,688\,000$ cu. ft. annual draft = 3.2 cu. ft. per sec.

Where a number of studies of a stream are to be made, a set of curves applicable to the given storage constant and *CV* may be computed as in Table 3.

Column (6) of Table 3 may be calculated from the 95% curve (Fig. 13). Thus, for 1.0 draft the storage is $7.4 \times CV$, or $7.4 \times 0.16 = 1.19$. This represents the annual storage in terms of the average available run-off, which is 0.541. The required annual storage is, therefore, $1.19 \times 0.541 = 0.640$. The next draft

represents 0.99 of the available run-off, or $1 - 0.0625\ CV$, and for this draft Fig. 13 shows a storage of $5.2\ CV$; then $5.2 \times 0.16 = 0.832$, which multiplied by the available run-off of 0.541 gives an annual storage of 0.452.

TABLE 3.—COMPUTATION FOR 95% YEAR STORAGE-DRAFT CURVE.

0.1 tributary ratio; $C\ V = 0.16$. 60-day constant; 95% annual run-off (Fig. 12) = 0.768.
Maximum possible draft (Fig. 24) = 0.541.
Limiting draft for seasonal storage = $0.541 \times 0.768 = 0.415$.
Drafts of less than 0.415 follow the 95% seasonal storage curves (Fig. 6), corrected for 60 day storage constant.

Draft, in terms of maximum possible. (1)	Draft, in terms of average run-off (2)	Seasonal storage. (3)	Annual deficiency, Column (2) — 0.415. (4)	Monthly storage, Column (3) — Column (4). (5)	Annual storage. (6)	Total storage, Column (5) + Column (6). (7)
1.00	0.541	0.167	0.126	0.041	0.640	0.681
0.99	0.536	0.164	0.121	0.043	0.452	0.495
0.98	0.530	0.160	0.115	0.045	0.355	0.400
0.96	0.519	0.154	0.104	0.050	0.261	0.311
0.94	0.508	0.148	0.093	0.055	0.169	0.224
0.90	0.487	0.136	0.072	0.064	0.095	0.159
0.85	0.460	0.121	0.045	0.076	0.052	0.128
0.80	0.433	0.107	0.018	0.089	0.018	0.107
0.768	0.415	0.098	0.000	0.098	0.000	0.098

TEST OF THE CURVES

This whole discussion rests on several assumptions which are not easily susceptible of proof, while the work of making tests with long-term flow records is very laborious. In making such tests daily flows or average weekly flows should be used, since monthly flows usually will not show the full quantity of water flowing in the main stream at rates exceeding the draft. Some tests have been made using a 20-year flow record of the Hudson River at Mechanicville, plotted in weekly averages. The assumptions as to the tributary ratio are arbitrary and do not refer to any particular reservoir locations. The computed CV is 0.16, and storage constant 60 days, thus necessitating interpolated drafts from the tributary storage curves (Figs. 25 to 33).

1.—The assumed reservoir capacity = 0.46 of the average run-off at the point of use.
The assumed average run-off at the point of use = 1.0.
The assumed average run-off at the reservoir = 0.2 = tributary ratio.
The 95% regulated draft (computed) = 0.65.
Result: The draft can be maintained 18 years, or 90% of the time. There are deficiencies of 0.16 and 0.128 in two of the years and the reservoir is from 95 to 99% empty during several other years.

2.—The assumed reservoir capacity = 0.68.
The assumed tributary ratio = 0.1.
The 95% regulated draft (computed) = 0.54.
Result: This is the maximum possible draft for these extreme conditions. The same draft with the reservoir at the point of use would require only 0.15 storage, which indicates the large quantity of annual storage required because of the location of the reservoir. The draft is actually maintained 19 years, or 95% of time.

3.—The assumed tributary ratio = 0.5.
The assumed draft = 0.87.
The 95% storage required (computed) = 0.60.
Result: The draft is maintained through the 20 years of record, but an actual storage of 0.58 is needed in the twelfth and again in the fifteenth years.

4.—The assumed tributary ratio = 0.1.
The assumed draft = 0.5.
The 95% storage required (computed) = 0.20.
Result: The draft is maintained 70% of the time, with deficiencies of 0.035, 0.029, 0.045, 0.002, 0.012, and 0.088, respectively. One group of three dry years, during which 0.26 storage would have been required, causes the failure to maintain the draft. One of these, the year 1910-11, was extremely dry, the run-off being only 68% of the average.

It seems to the writer that these tests tend to confirm the reasoning on which the tributary storage diagrams are based. They are bound to throw some light on the subject. It is hoped that other engineers who may have occasion to study the question, will subject the data given herewith to like critical tests.

Conclusion

Although duration curves, plotted from flow records are much used in estimating the probable future occurrence and magnitude of stream flows, there are comparatively few American streams which have been satisfactorily gauged for more than ten or fifteen years, and in many instances the engineer is fortunate if even five years of recorded flow are available for his guidance. It is to be regretted that work of this character, of such real utility, does not strike the popular fancy so that adequate appropriations may be made available.

Conclusions based on such short-term records, unless supplemented by additional data, may be considerably in error. The probability curve furnishes a means of extending such records in a most satisfactory manner although it has not received general recognition. Mr. Hazen's "seasonal storage" curves are probability curves, not of flow, but of deficiency based on flow, and they provide simple, rapid means for predicting the amount of storage necessary for release in dry years to maintain a desired use or draft. For all rates of flow and for all percentages of time, only one simple calculation is needed, which may be based on a short-term flow record.

The coefficient of variation, which is the key to the total storage requirements for high rates of draft, likewise may be determined from a short record with small expenditure of labor.

Both these constants for a given stream are based on the usual variations rather than on those extremes which occur at long intervals. It is important to note this fact, for to it is due the accuracy obtainable with short records by the method here advocated. Having computed these two constants all problems of deficiency and storage may be solved by reading from the diagrams here given, with results always consistent and permitting the economic

reservoir capacity or location to be determined with the same ease, for example, as its cost.

The total storage curves presented are not of universal application because they are computed for a coefficient of skew of 0.6, which is suitable only to streams having a *CV* of 0.3, or less. This was shown by H. Alden Foster, Assoc. M. Am. Soc. C. E., in his paper, "Theoretical Frequency Curves and Their Application to Engineering Problems".* A range of *CV* from 0.0 to 0.3 seems, however, to cover most streams of the Eastern United States, and perhaps many other localities.

In his paper, Mr. Foster also shows that while the *CV* may be found with a fair degree of accuracy by means of a short-term flow record, a definitely accurate value of the *CS* can only be ascertained through the use of a record of considerable length. Although further computations of the *CS* for streams in various parts of the world would be of great value, the error in the amount of storage required as based on a somewhat inaccurate *CS* need not be large, and it is possible that only two or three values of *CS* need be selected as a basis for a series of storage-draft curves of practically universal application. The test of annual storage required for the Murray River, having *CV* of 0.52, given by Mr. Foster,† furnishes some ground for this statement.

There is abundant opportunity and room for further research into the apparent vagaries of rainfall, run-off, flood flow, and other problems of hydrology; and certainly there is much to be gained thereby. A comprehensive examination, classification and study of the most reliable long records of this and other countries, as to their coefficients of variation and skew, their curves of flood flows, etc., while involving considerable labor, would surely prove of immense value in this age when stream control for irrigation, water supply, flood prevention, navigation, and power is of such immediate interest.

The author wishes to thank William Barclay Parsons and John P. Hogan, Members, Am. Soc. C. E., who, to use a popular expression in a literal sense, "engineered" the New York Water Power Investigation; and whose encouragement and interest made this study possible. In the endeavor to find a logical basis for the work and in order to check the resulting curves a large amount of tedious and not always interesting computation was performed by Messrs. H. Alden Foster, H. R. Bouton, J. E. Beswick, and F. G. Bennett, Associate Members, Am. Soc. C. E., and C. A. Wright, Jun. Am. Soc. C. E., whose intelligent and patient co-operation is sincerely appreciated.

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

† *Loc. cit.*, p. 169.

A CENTURY AND A HALF OF AMERICAN ENGINEERING

ADDRESS AT THE ANNUAL CONVENTION IN PHILADELPHIA, PA., OCTOBER 5, 1926.

By George S. Davison,* President, Am. Soc. C. E.

A great nation is viewing retrospectively its life of a century and a half, and taking an inventory of its achievements. The main factors in its growth lie in its area, population, and wealth. The Continental United States of to-day covers ten times the number of square miles within the borders of the original thirteen States. Its population has increased from about 3 000 000 to 115 000 000. No means are at hand for estimating the wealth of the nation on its natal day, but this much is certain—it was insignificant in comparison with the $400 000 000 000 that stand to its credit to-day.

It is eminently proper that members of the American Society of Civil Engineers as citizens of such a nation should gather here in Philadelphia, where its birth is being celebrated, to attest their interest in its affairs and their loyalty to its aims and purposes. It is fitting that while here members of a Society the life of which has covered half that of the United States, should inquire what part the Civil Engineer has played in the creation of those conditions which, within one hundred and fifty years, have transformed America from a few struggling colonies into a leading nation of the world. We are using the term "civil engineer" in its broadest sense, in order to include all those who through scientific processes are directing the great sources of power in Nature that they may be most useful to mankind. We recognize that the mechanical engineer, the mining engineer, the electrical engineer, the hydraulic engineer, the sanitary engineer, or any engineer of special calling, is only practicing such a branch of civil engineering as his title would indicate and his professional knowledge justify.

This does not mean that to the engineer alone belongs all the credit for the colossal advances we now enjoy as a people. His efforts have been involved with those of the artisan, the industrial manager, the inventor, the capitalist, the statesman, and many others; but it is his ability "to design as well as to direct engineering works" that must be relied on to bring successful results not only out of his own vision but out of the vision of others, and to cause the dreams of man to come true.

The inhabitants of that part of America which comprised the original thirteen States were mainly of English, Dutch, German, and French descent, whose forebears had left their native lands either in search of adventure or

* Pres., Gulf Refining Co., Pittsburgh, Pa.

to escape conditions that were intolerable. Even had they possessed the knowledge of the arts and sciences as of their day and of the place from whence they came, their new surroundings offered no encouragement to make use of such knowledge. They had anticipated the simple life mingled with hardships; and they were not disappointed. Their existence resembled camp life, with little idea when they would move and where next they would go. Temporary expediency answered every purpose. Labor-saving devices, even had they been conceived, could not have been enjoyed because of a lack of capital with which to develop and install them. Their ambitions were confined to securing for themselves food, shelter, and clothing. To till the soil, to fell timber, and to weave the cloth were their main tasks. Industry had developed in only a few directions. It might be remarked that one of these was the production of iron, which was smelted with charcoal as fuel.

The territory occupied lay along the sea and extended less than 100 miles toward the Appalachian Range. Commerce sought the sea. Inland transportation either by land or water was inconsiderable.

The manufacture of power as an essential for rail transportation and industry had not yet been initiated. In fact, at that early time, the untamed wind, the fall of waters, and the beast of burden were the only supplements to manual labor. Material advancement from this condition was long delayed. It was not until 1830 that the steam railroad came, and even twenty years later that great agent of civilization had scarcely begun to play its part. Turning, then, to the object of our inquiry, namely, what has the civil engineer accomplished, we shall find his greatest triumphs within the past seventy-five years.

The early pursuit of the people was agriculture. Land was the first consideration. Its area was to be determined and apportioned. Therefore, land surveying early engaged the attention of the engineer. It was in 1824 that the first civil engineering school in America was opened, twenty years after the establishment of the United States Military Academy. The founder of this school, Stephen Van Rensselaer, a man of affairs, served as a member of the Erie Canal Commission from its inception in 1816 until his death in 1839. His contact with the problems of that time had given him a vision of the influence that the technical mind would have over the future affairs of the people. In stating his object in founding this institution, he said:

"I have established a school for the purpose of instructing persons who may choose to apply themselves in the application of science to the common purpose of life."

The first class in civil engineering was graduated in 1835.

These facts show what little was being done one hundred years ago to educate a young man for the profession; from which it may be deduced that the demands then made on the profession for service were few in number and simple in character. The story of the country's progress in applied science in its first half century confirms this view.

At that time transportation was—as it has continued to be—the great problem underlying the prosperity of the nation. Then it dealt with sailing

vessels and horse-drawn vehicles. Later, the highway yielded to the canal, then the canal to the railway, and now the railway is viewing with alarm the return of the highway to public favor.

The beast of burden has all but disappeared, and human energy has been replaced with mechanical power from Nature's great storehouse. That public service may be brought to the home, the switch, the valve, and the spigot have been installed. That worthless land may be redeemed, irrigation and drainage have been effected. That the worker may be comfortably sheltered and profitably employed, the skyscraper and the industrial plant have been built. That commerce may move whither it will, the face of Nature has been smoothed with bridge and tunnel.

Relation of Various Engineering Fields

Passing now from the general to the particular, it may be instructive to present a brief historical review of the more important pursuits in which the civil engineer has specialized. To deal with the many subjects in anything like a chronological order is difficult, as the various branches of civil engineering mingle indiscriminately both as to time of development and scope of work. Such, for instance, is Surveying, the first topic treated; certainly it was one of the earliest forms of engineering in America and it still has a large and important following among engineers. Surveying is, so to speak, a common factor in all engineering work.

In order of development the agencies of transportation come next, namely, Waterways, Highways, and Railways, in order. Logically, the study of Power succeeds these for it is a *sine qua non* for all transportation. Following hard on the development of transportation came a variety of attendant engineering pursuits, as Ship Building, Bridge Building, City Planning, and Tunneling.

With the growth of the city a number of vital needs presented themselves. Especially did the community look to the engineer for the protection of health, and of him was demanded the development of Water Supply, and of Sewerage and Sewage Disposal. Likewise, with the concentration of urban population, the need of housing and industrial development inspired the great field of Structural Engineering. In due season, agriculture like industry, demanded expansion, economy, and conservation; thus as one of the later, indeed almost the latest, great National engineering development came Irrigation.

In this order, more or less logical, will be treated the more important events in the various engineering fields during the last one hundred and fifty years of American history, each topic under its own appropriate heading.

Surveying

The art of surveying, like other processes of engineering, has made important progress during the last century and a half. It is indeed a far cry from the rude compass and chain used by the Father of His Country to the delicate aerial cameras and submarine sounding devices in use at the present day.

An epochal event in the development of surveying occurred toward the end of the Eighteenth Century. Ramsden, an English instrument maker, per-

fected a dividing engine with which he produced a theodolite having a circle 36 in. in diameter and capable of being read to a single second of arc.

The Nineteenth Century has been notable for the introduction of the steel tape to take the place of the old surveyor's chain, the use of better and more precise methods for various kinds of surveying work, particularly geodetic surveying, and the development of special instruments.

Even as late as 1887, the chain was spoken of as the usual instrument utilized in land surveying and the steel tape, said to have first been made from hoop skirt wire, was considered only a means of accurate measurement. It was an expensive device. Modern manufacturing methods, together with its superior accuracy, have made it the standard instrument.

In geodetic surveying, bars of various materials, two in number, used alternately with careful alignment, were employed to measure primary base lines. It was a very expensive and not a highly accurate method. More recently, the invar tape with a coefficient of expansion roughly one-thirtieth that of steel, has made base-line measurements, with an accuracy of 1 in 1 000 000, possible under almost any temperature conditions.

As indicating the extreme accuracy and ingenuity of modern surveying devices for special purposes the latest aneroid barometer which exhibits readable variations for a difference of a few inches in elevation may be mentioned. Perhaps the greatest skill applied to surveying methods at present is being given to aerial surveying. Wonderfully accurate cameras have been developed. Some of the devices for reducing the photographs to engineering maps are even more remarkable, including a stereoscopic measuring table, permitting contours for areas of high relief to be drawn on the photograph to an interval even as small as 10 ft.

A system for surveying public lands was authorized as early as 1784; it has been since modified, to define the locations of principal meridians, standard parallels, township lines, and section lines. This work is in charge of the General Land Office.

The United States Coast Survey, although authorized in 1807, was not instituted until 1817, while material progress was not made until about 1832. Commencing on the Atlantic Coast, the work eventually included the Gulf of Mexico and the Pacific Coast. About 1870, authority was given to connect the surveys of the two coasts. These labors involved various geodetic determinations which gave an added significance to the work of the Survey, so that in 1878 it became officially the "Coast and Geodetic Survey" under which name it now operates.

Somewhat akin to this Government Bureau may be mentioned the United States Geological Survey, which, however, restricts its activities to the inland territory of the United States. The Geological Survey is the outgrowth of several separate activities under various Government departments which were merged in 1879. Besides other duties the Geological Survey makes topographic surveys and maps, geological maps, and hydrographic surveys of the inland waters of the United States.*

* *Transactions*, Am. Soc. C. E. (1905), Vol. LIV, Part B, p. 419.

One of the most recent developments for taking soundings through water is called the Sonic Depth Finder. This instrument, attached to a vessel, catches the sounds reflected from the bottom of the ocean and by delicate measuring devices enables the depths through which the sound has passed to be measured.

Of modern surveying methods that from the air bids fair to outstrip all others for reconnaissance work, for map revision, and for securing the flat map features in extensive topographic work. A series of photographs taken from an aeroplane flying at a known height is made inclusive enough to cover the entire area with sufficient overlapping. A ground survey covering objects and elevations capable of identification on the air photographs links the picture to the actual ground topography. This method is in frequent use and under rapid development.

Little would the surveyor of one hundred and fifty years ago have dreamed that such a rapid and reasonably accurate method were possible. Thus has modern engineering science succeeded in utilizing all the vast improvements in methods to make the profession still further a benefit to humanity.

Waterways

Prior to the founding of our Government and for some time thereafter no effort was made to improve for navigation purposes the inland waterways nor the inlets and harbors bordering on the sea coast. In the course of events the United States Government assumed control of all natural watercourses. In recent years, with the expenditure of monies taken out of the general funds of the nation, it has deepened and otherwise improved the channels leading into deep-sea ports, and the more important rivers.

A long time prior to these improvements, however, private capital took up the question of developing the natural resources and expanding the trade of the country by means of artificially constructed waterways. The earlier plans with a few exceptions were confined to reaching near-by markets with the products of mine and factory. More ambitious projects than these, involving extensive mileage, were necessary for general trading purposes.

The first long-distance canal was the Erie Canal completed in 1825. This, together with the Hudson River, connected the Great Lakes with the Harbor of New York and assisted materially in extending trade with the seaboard into Northern Ohio and a considerable area contiguous to the Great Lakes. Once this territory was reached, at least three important canals were built from points on Lake Erie to the Ohio River, thus providing an all-water route from the Hudson and Delaware Rivers to the Mississippi Valley. This circuitous way was necessary to get an all-water route around the Appalachian Range. Meanwhile, the State of Pennsylvania developed a route across the Allegheny Mountains, thus shortening the distance from tide-water to the Ohio River by one-half. This artery of travel was a combination of railway and canal, with inclined planes between Hollidaysburg and Johnstown, by means of which the loaded canal-boats, when separated into sections, could be carried over the mountains.

Canal building extended through a period of about thirty-five years, or until 1850. It has been estimated that in the era of artificial waterways 4 500

miles of canals were built at a cost of $214 000 000. Much of this mileage was doomed to financial failure before it was finished, because meanwhile railroads had been inaugurated in the United States (1830). Once their advantages were realized the death knell of those canals serving the interior was sounded, so that even now the channels and locks of the greater portion of these canals have been entirely obliterated.

The only important canal that has survived the competition of the railroads is the Erie Canal. It is State-owned and has undergone a complete reconstruction to accommodate boats of larger tonnage and deeper draft, to be operated by steam. Even with this modification the freight carried is inconsiderable in comparison with the traffic on the railroads paralleling it, its tonnage for 1924 being equal to about 60 000 carloads.

However, even since the advent of railroads there have been examples of the canalization of important streams. Some of these earlier projects were initiated with private capital, but, because of the policy of the Government during the past thirty years to own and operate such works, these ventures have been taken over through condemnation proceedings. Under the general program of serving navigation, the canalization of rivers is continuing under the direction of the War Department. In the case of some more important rivers, such as the Mississippi, the improvements are under the jurisdiction of Commissions, organized under Federal acts.

A most important part of the program of the Government in the interest of navigation deals with the improvement of the harbors along the coast and on the Great Lakes for vessels of deep draft. Since 1902, this work and that relating to rivers has been carried on under the direction of the U. S. Army Engineers, as provided in the River and Harbor Act. The enormous work accomplished since the passage of this Act in 1902 may be realized when it is understood that for these purposes fully $500 000 000 has been spent.

Great as may appear the sum spent by the Government in the interest of navigation, it is far overshadowed by the enormous amounts expended by local harbor authorities and private interests to provide docks and equipments to handle the traffic at terminals. The cost of the grand total of all governmental and private construction in the interest of waterways runs into billions of dollars, the greater part of which has been accomplished within the past fifty years.

Beyond all this special mention should be made of the Panama Canal, constructed within the years 1907-14 and costing $367 000 000.

In addition to carrying out this vast construction program, to the engineer is committed the responsibility of maintaining these great works so that trade and commerce may be served to the highest degree of effectiveness.

Highways

Like Topsy, the earlier roads in America "just grew". As communities became active centers of business and as local governments began to function, inexperienced supervision assumed the responsibility of maintaining these highways, to which only such attention was given as would keep them in a barely passable condition. In the course of time companies were organized for

the purpose of assuming charge of the more important routes, and they charged toll for the service rendered. Meanwhile, canals had demonstrated their advantages over roads, by furnishing better service at a lower cost; but soon both fell into disfavor on the advent of the steam railroad.

The encouragement of a government liberal with its subsidies to those who would construct railroads in virgin territory, and the prospects of a rich return on traffic already created by the former methods of transportation, made investments in the new field so attractive that capital for building turnpikes "dried up". Prior to the fall of the highway before the irresistible advance of the railroad, the Conestoga wagon had been the freight car, the stage served as a passenger coach, and the Pony Express as the fast mail.

The highway, however, ceased to be an empire builder and the turnpike companies settled down to bide their time hoping for a better day to dawn. In due course the delay was rewarded, in most cases by condemnation proceedings, through which the public compensated the stockholders of such companies.

Early road building afforded little opportunity to the engineer. It was only at a very recent date that his services became necessary in highway construction. Forty years after he had taken over the building of railroads, the machine which at first had the appearance of a plaything became a demonstrator of the advantages of good roads. I refer to the bicycle. By 1890, it had millions of devotees, who, through the League of American Wheelmen, demanded better roads of the State and local authorities.

Although this gesture made a lasting impression, the bicycle did not enjoy its popularity long enough to secure prompt results. Better arguments for good roads came with the motor car. In 1900, motor vehicles to the number of 8 000 were registered in the United States; in 1910 there were 468 000; and in 1925 there were about 20 000 000. While the operation of motor cars was at first confined to city streets, in due course of time the desire to use the country roads could not be suppressed and the main highways were improved to meet the demand of the motorist.

The internal combustion engine has wrought a miracle and the automobile has settled down for a long visit. What at first was a luxury has become a necessity. Even the farmer, the clerk, and the laborer must have his pleasure car. Auto-buses have increased travel over important highways and built up new business over routes that were almost abandoned. Commerce finds the massive truck suited to its purpose in the transportation of freight. The railway that once so ruthlessly robbed the highway of its good name, is now seeking mercy at the hands of "good roads".

To meet the advancing popular needs dustless roads were developed—usually by surface treatment of the existing macadam. In due time bituminous macadam and bituminous concrete were evolved and contemporaneously the cement concrete surface. However, with all these improvements highway engineering has had the utmost difficulty in keeping abreast of the increasing traffic demands.

There are 3 000 000 miles of roads in the United States. Of this, at the end of 1924 much had been improved—of low type construction, 308 000

miles; of macadam, 115 000 miles; and of high type roads, 46 500 miles, a total of 468 500 miles. On this mileage there had been spent about $1 000 000 000. In 1924 alone 31 540 miles of roads were improved. The civil engineer may view the situation with complacency, as the highway will continue to require his services for some years to come. His specifications of five years ago for road building did not anticipate the heavy wheel loads and congested traffic that must now be met. To-day, he is using better material, deeper foundations, more scientific drainage, and heavier and more enduring bridges. What was only recently "good practice" soon had to be renewed.

While the engineer has been building this great network of highways, he has been designing the cars that will use them and, in turn, finding the fuel to run the cars.

Indeed, modernizing the idea in an old nursery rhyme, it may be said:

"This is the road that Jack built,
This is the car of Jack's design,
That sped over the road that Jack built,

"This is the gas that Jack produced,
That was used in the car of Jack's design,
That sped over the road that Jack built."

The great strides that are being made in road building, and in the manufacture of vehicles to travel on them, is strictly due to the civil engineer, and it is limited only by the amount of money which the public is willing to spend in their improvements. However, it must not be overlooked that this great task on which the engineer is engaged has already rendered useless millions of the capital which he spent for other clients in former years. The buses *de luxe* have thrown the interurban line on the scrap heap, and have struck a serious blow at the steam railroad with respect to short-haul of passengers and freight.

Railways

Industrial railways—horse-drawn—preceded the steam railroad in America, and that as early as 1806. The steam railroad came into existence in 1830 with a length for that year of 30 miles. The existing system of 250 000 miles is the country's greatest asset and its present state of efficiency constitutes the proudest achievement of the engineer.

The fact that the weight on the driving wheels of the locomotive has increased steadily from less than 20 tons in its earlier years to nearly 400 tons is the answer to the question of what single movement has had the greatest influence on our prosperity. These heavier locomotives have been followed by cars of greater capacity and heavier rails of improved quality. Roadbed and structures have been re-designed to meet the greater stresses imposed on them. Main tracks have been paralleled and terminals enlarged.

The handling of longer trains and an increased number of them by these more powerful engines has been made possible by the introduction of the air-brake, the automatic coupler, the automatic safety signal, and, now, of train

control. The safety, speed, and comfort of passenger travel have been advanced through these same improvements and in addition by the introduction of the palace car, of the vestibule platform, of steam heating and electric lighting of coaches, and of stations of monumental design and great utility. Coincident with these betterments, grade crossings have been abolished, grades and curvatures reduced, alignments improved, locomotive fuel efficiencies greatly increased, and electric traction installed where it was necessary or desirable.

It is a matter of record that in the olden days of horse-drawn vehicles the cost of moving freight over improved turnpikes was about 20 cents per ton-mile, and the rate per ton-mile on canals was not less than 3 cents. For at least twenty years before the general advance to higher costs occasioned by the World War, the average revenue per ton-mile on all railroads in the United States was less than 0.75 cent. The increase in advantages to the traveling public over the days of the stage coach has been in speed and comfort rather than through reduction of rates of fare, although fares have undergone a revision downward.

Granite blocks were first used as a foundation, on which the rails were laid either directly or with intermediate stringers. Soon, however, the present method of supporting the rail on cross-ties was adopted.

The first rails were of wood, protected on the wearing surface by iron straps. Iron rails came into use as early as 1835. They were of the general form of the T-rail of to-day. The first ones, imported from England, were in 15-ft. lengths weiging 36 lb. to the yard. The first iron rails rolled in America appeared in 1845, at which time the weight had increased to 56 lb. and the length to 30 ft. Steel rails were imported to a small extent about 1860. American steel rails were produced by the Bessemer process as early as 1865, and by 1880 most of the original iron rails had been replaced by steel, the maximum weight being 67 lb. The present general form of splice-bar joints has been used for at least fifty years. Meanwhile, the weight of the rail has advanced to a maximum of 130 lb. and open-hearth steel has gained favor over the Bessemer product.

The first locomotives were wood burners from actual use. Coal was used as early as 1855. The railroads penetrating the Southwestern and Pacific Coast territories have been using oil for fuel for twenty-five years. Within that same period, the electric locomotive has been adopted for handling trains in and out of certain busy terminals, particularly where smoke from steam locomotives would not be permissible. Some studies have been made looking to a more extended adoption of this form of power, and we can look forward to many important developments in this practice. Following the idea that where the necessary water power is available the generation and distribution of hydro-electric power may show greater economies and adaptability to the service required than steam power, about 450 miles of one great railroad have been electrified.

The passenger coach of the early days of railroading would accommodate a mere dozen passengers. There were as many patterns of coaches as there were railroads, but by 1840 the coach had assumed its present form, mounted on

center-pin trucks. (The railway truck, incidentally, is an American invention.) The length of the body was 40 ft., the width 8 ft., and the capacity about 20 passengers. The early coaches were heated by stoves, at first burning wood, and, later, coal. At night their interiors were lighted with candles. On his rounds the conductor found his way about the dim car with the aid of a hand lantern. The coaches were of wood, and frequently in cases of wreck they caught fire from the heating systems, cremating passengers who might otherwise have escaped.

The coach continued to increase in size to the present length of 80 ft. and width of 10 ft., in weight to 122 000 lb., and in seating capacity to 84 passengers. The stoves have been superseded by steam heat from the locomotive. The method of lighting has passed through the stages of kerosene lamps, gas (stored under pressure in tanks beneath the floor), and, finally, electric current, produced at first by a steam engine located in the baggage compartment and more recently by generators belted to the axle of the coach. The water cooler in the end of the coach is another American idea, which deserves honorable mention because of a modest and faithful career that has not been properly appreciated.

Efforts to furnish sleeping quarters for passengers were made as early as 1836, but the real Pullman did not make its appearance until 1860; and, in 1867, there were just forty-eight of them in service. The first diners were called "hotel cars," and consisted of a kitchen installed in the end of a sleeping car. Although the present form of the dining car followed shortly, the call of "twenty minutes for dinner" was still heard throughout the land until about twenty-five years ago.

The air-brake was first applied to a passenger train in 1868, just preceding the introduction of automatic couplers. It has undergone marked improvement both in principle and effectiveness since its original installation.

The first railroad freight car had a capacity of only a few tons. Its development as to running gear, couplers, automatic brakes, and increase in dimensions is similar to that of the passenger coach. To-day, its maximum carrying capacity is 70 tons. At first, freight cars were loaded and unloaded by hand. Many improvements have been made by which such labor has been greatly reduced. The grab bucket and the "hopper bottom" are examples. An entire car, loaded with coal, may be run into a cage-like framework that is revolved transversely to discharge the contents of the car.

The stage and the canal-boat moved at the rate of 4 miles per hour. The limited trains of to-day carry the passenger, the letter, and the express package at a speed of 50 miles per hour. The economies in time and money resulting from the higher speed are not susceptible of calculation.

The horse car for urban service was a novelty seventy years ago. A few decades later the cable car was evolved as an improvement. It was believed that besides increasing the capacity of surface lines and providing quicker service, great economies could be effected by a mechanically operated system with the power developed at a central station and distributed by a moving cable. The cable car, however, was quickly abandoned for the trolley car

except on streets of steep grade where it is still in use because of its high degree of safety. When electric-current transmission by feed wire with application through motors on the car was introduced, it sprang into favor quickly, and a whirlwind construction program was followed until fifteen years ago. This movement wiped out the horse car, and, in many cases, seriously affected the local passenger business of the steam railroad.

Fifty years ago, the congestion of vehicular traffic on city streets suggested the idea that part of it must be transferred elsewhere. Elevated lines were constructed on which multiple-car trains hauled by dirty, noisy, and jerky steam locomotives were operated. While this development enhanced property in the vicinity of the stations, the damage resulting to property abutting on such elevated lines was so enormous that they failed of general adoption. A quarter of a centry ago the subway idea was advanced as a better method of mass transportation for the reason that it serves the public equally well and does not have the same blighting effect on adjacent properties.

Power

To describe power development in the past one hundred and fifty years is in effect to review it from the age of the guilds. During that time revolutionary strides have been made and such undreamed-of applications have proved successful that there is scarcely one of the devices in use a century and a half ago that has not become obsolete.

The first use of steam, which has been the principal prime mover of the Nation's industries, was the foundation of man's present freedom from toil—the beginning of our present mode of living. One hundred and fifty-two years ago, James Watt, the mathematical instrument maker of Glasgow, perfected the reciprocating steam engine. Oliver Evans, the American stepfather of the steam engine, and the earliest of the New World's great engineers, first advocated the high-pressure steam engine.

For nearly a century the steam engine was the undisputed source of mechanical power in America. With respect to the operation of factories, it abolished the inflexible water-wheel and windmill. The steam engine became known in time to create a system of transportation theretofore practically unknown—the railroad; it raised the floating commerce of the seas above the hanging helplessness of sails.

A climax in the building of steam engines was marked by the one installed to supply power for the Centennial Exposition held in this city (Philadelphia) in 1876. This gigantic vertical engine weighing 700 tons, a Corliss machine of 1 400 h.p., was, in its day, the last word in American steam engineering.

At this same Exposition was displayed a then novel form of energy—electricity. It depended on steam or water power for generation, but, once made, could be transmitted easily by wires—no belts, gears, or rope drives were needed. The prime mover could thereby enlarge the field through which its energy might be distributed.

One hundred years after Oliver Evans' first compound engine, the steam turbine appeared. Its high speed proved to be ideally adapted to the generation of electricity. Later, the Pelton water-wheel, an impulse turbine,

was connected with the dynamo and thereby long neglected water-power sites were harnessed and their power spread by electricity to industrial centers. Electricity is now so easily transmitted, so simple of control, and so powerful in work, that two-thirds of the Nation's machinery is directly operated by electrical power.

Another progressive step in the flexible distribution of power has been made through the use of the air compressor.

The past twenty-five years has witnessed a wonderful expansion in the building and use of the internal combustion engine. It has found its principal field in transportation. By automobile, motor-boat, and aeroplane, the passenger may be his own engineer and pilot as he journeys by land, by water, or through the air.

A simple method of measuring the advance of the Nation in recent years is to note that in 1900 it generated 13 000 000 industrial h.p. and that in 1925 the quantity had soared to 55 000 000 h.p. In 1900, it utilized 2 500 000 000 kw-hr. of electricity; in 1925, this had been increased to 68 000 000 000 kw-hr.

The development of power in the United States is one of the marvels of all times. Its wide use conserves the individual's energies and shortens his working hours, gives him higher wages through quantity production, supplies him with many articles of living never before available for general distribution, and, in short, raises his standard of living above that of rulers of other centuries.

Ship Building

Ship building in America is as old as Jamestown and older than Plymouth. Every Colony had its shipyards. Prior to the Revolution the annual launchings had reached 27 000 tons—Gloucester fishing schooners, deep-sea whalers, and coastwise traders made up the Colonial fleet. Commerce with the outside world was handled generally in foreign bottoms.

In 1781 a merchant ship, the *Alfred*, was acquired by the United States Government and was constituted the first (and only) ship of the Navy. John Paul Jones served as First Lieutenant on this vessel. The *Alfred* was of 220 tons displacement; to-day, the size of the Navy is 2 253 000 displacement tons. As compared with the *Alfred*, the battleship, *California*, is of 33 190 tons displacement.

In 1791, Alexander Hamilton, Secretary of the Navy, reported an American Merchant Marine of 476 274 tons. The Merchant Marine of to-day comprises 14 878 761 tons.

The *Clermont*, a vessel of 160 tons built by Robert Fulton in 1807, was the first steam vessel. Fulton also, in 1814, built the first war vessel to be propelled by steam—a ship of about 1 200 tons. The first steam vessel to cross the Atlantic was the *Savannah* (of 350 tons) which sailed on her maiden voyage in May, 1819, from the Southern port for which she was named. In 1843, the first iron ship in America was built.

The *Great Western*, probably the first English steamship designed especially for the trans-Atlantic trade, created a sensation when she arrived in America

on her maiden voyage in 1838. She was of 1 340 tons and required 15 days to make this ocean passage. As an evidence of progress, it should be stated that there is now being built an American vessel of 22 000 tons, the speed of which will be such that she can make the trans-Atlantic trip in 7 days.

Fulton, in building steam vessels, used paddle-wheels about 15 ft. in diameter, operated directly by the connecting rod of the engine. The walking-beam method of transmitting this power is just one hundred years old, and still maintains its popularity for inland navigation. The screw propeller, invented by John Ericsson, was first applied in the United States to the Steamer *Princeton* in 1843. It is now the accepted method of propulsion for vessels, especially on the high seas.

The change from coal to oil for boiler fuel in large merchant and naval ships began in 1900. American practice in handling the steam from boilers to its point of application successively passed through the stages of the simple engine of Fulton, and the compound engine of seventy years ago, to the triple-expansion engine, now about fifty years in vogue. Twenty years ago the direct-drive steam turbine began to find preference over the reciprocating steam engine, for certain types of vessels. One of the latest developments with respect to power for ships is the internal combustion engine using oil for fuel; and the very latest device for the application of marine power is the electric drive, with a choice of steam turbine or Diesel engine as the prime mover.

Admittedly, the United States is not in the first rank in the matter of a merchant marine, although she is fully up to standard in naval equipment. She is well supplied with tonnage plying coastwise and on the Great Lakes. Since the days of Robert Fulton her inland waterways have been routes for transportation. The difficulty in maintaining uninterrupted traffic on some of these streams and the competition of the railroads, which were better and quicker, caused the abandonment of much of this service. However, while these waterways have been passing through periods of uselessness, Congress, with the advice of the War Department, has been providing funds for revetment and canalization on them. The engineer has been engaged for years on such works, and now, with improved channels on the one hand and a greatly increased cost of rail transportation on the other, inland water transportation is again coming to the fore as one of the important assets of the nation.

Bridge Building

The art of bridge building is believed to be as old as civilization, but the science of it could not have antedated the first demonstration of the law of gravitation. Empirical rules and judgment based on experience were the guides in early construction.

The masonry arch is one of the oldest forms of the bridge. Besides serving a useful purpose, it afforded the early engineer a means of expressing his artistic taste. Primitive America was forced to sacrifice the beauty and durability of the arch for designs of less costly material. The first stone arch in this country was built in 1800, and, although devoted to public use, its cost

came from a private purse. Beginning with 1820, famous aqueducts to carry water supplies to several of the larger cities were constructed. With the advent of the railroad, a limited number of arches were included in its original construction. Later, many large railroad bridges were renewed in stone.

Until seventy-five years ago the question of cost determined to what extent the masonry arch should be used. Then dawned an era when communities with funds available to meet their asthetic desires returned to the ancient idea of giving utility an artistic finish. A large number of these structures may be found in American cities, the combined work of the engineer and architect.

Continuing to follow both the useful and the esthetic in the arch bridge, cement concrete began to take the place of stone about thirty years ago, and in a short time stone was used only in the voussoirs. Later, the introduction of steel reinforcement reduced the weight and cost of arch bridges without impairing their strength, so that the use of stone is now limited almost exclusively to decoration.

The earlier bridges in America were of wood. Prior to 1776 only a few had been built, their lengths comprising a succession of short spans (about 20 ft.). It is recorded that in 1792 a span of 175 ft. was constructed. Before the steam railroad became a factor, many bridges of that length, and even greater, were built by such designers as Timothy Palmer, Theodore Burr, and Louis Wernwag. In the longer spans, combinations of the arch and truss were used, and much ingenuity in design was displayed, the framing timbers being determined from a study of models and from experience gained by the failures of previous attempts.

An epoch in bridge engineering occurred in 1840 when William Howe patented the truss which has since borne his name. In it iron rods were used for vertical tension members. This simplified the connections at the joints and made the determination of stresses in members easy.

The late Squire Whipple, Hon. M. Am. Soc. C. E., took out letters patent in 1841 for his bowstring truss, in which cast iron was used for compression members and wrought iron for tension members. From that time iron bridges were much in favor and many types of trusses were developed and built, some of which were identified by the names of the engineers who first suggested them.

The scientific computation of stresses in bridge members was demonstrated in 1847. Without regard to the material used in articulated bridge structures, serious difficulties had presented themselves in the design and construction of the connections between the members. The Howe truss had eliminated this difficulty as to wooden structures. In metal structures, and particularly those of cast and wrought iron in combination, the solution of the trouble was much delayed. The pin-connected truss that was evolved about 1858, principally to meet the requirements for long spans involving difficult problems in erection, simplified the connections. After this form of truss had gained great favor there followed the famous "pin *versus* rivet" war, between American and English engineers, which lasted for many years. In recent years shop and field

facilities have so improved in workmanship and cost in this country that American practice has drifted definitely toward riveted construction.

The relative cheapness of cast iron made it the popular material for compression members for many years. Cast iron in highway bridge construction was long in vogue for architectural effects after its use in main members had been abandoned. The largest, all cast-iron, bridge in this country was built over the Schuylkill River, at Chestnut Street, in this city (Philadelphia) in 1864. It is still in use.

The failure of a cast-iron railroad bridge in 1876, accompanied by a serious loss of life, resulted in the discarding of this material in such bridges. A similar action followed shortly with respect to highway bridges. Construction in wrought iron throughout was the prevailing practice until open-hearth steel became available at a price and of a quality that made it more desirable than iron. However, the substitution of steel for iron had been increasing gradually before the wholesale adoption of soft steel. The first all-steel truss bridge in this country was built over the Missouri River, at Glasgow, Mo., in 1879. Crucible steel was used in the arched spans of the Eads Bridge, at St. Louis, Mo., which structure was finished about 1875.

Many problems arising out of the intersections of highways and railroads with navigable waters have been presented to the bridge engineer. Government responsibility over waterways demands that the channels be protected by the provision of large clearances. The suspension bridge, the cantilever, or the steel arch, is used where the approaches permit an elevation of the channel span that will provide the proper vertical clearance. When sufficient clearance under a fixed span cannot be secured within a reasonable combined cost for construction and damages, movable spans on a lower grade line must be used. In such cases a combination of the science of the structural engineer and the ingenuity of the mechanical mind is necessary. Up to a certain period the horizontally revolving drawn span, balanced on a center pier, was believed to fit any case. Spans that may be moved vertically are improvements on that idea; the popularity of these types is illustrated by the great number and variety now in use.

From the first iron truss bridge of 77 ft., built in 1840, the maximum length of spans has increased until in this year of 1926 the longest simple truss is 720 ft.

The first suspension bridge was built in 1801. It was a highway bridge, with a span of 70 ft. Chains were used as suspension members, on which was hung the wooden floor. In July, 1926, in this city (Philadelphia) there was dedicated a suspension span of 1 750 ft., the cables being of the usual built-up wire section.

The first cantilever bridge of importance was built in 1876 with a center span of 375 ft. The longest cantilever in the United States at this time is 1 182 ft.

The first important metal arch bridge was the Eads Bridge. Its longest span, 520 ft., was completed in 1874. The Hell Gate Arch Bridge, built in recent years (1912-17), is 1 000 ft. long.

City Planning

One hundred and fifty years ago there were only three cities in this country with any considerable population. Boston had no systematic layout; New York and Philadelphia were being developed on a rectangular basis. The New York plan was the result of the work of the first City Plan Commission in the United States, created in 1807; the basis was a mason's wire screen. It, however, was influenced by an earlier idea from Philadelphia. The Philadelphia design was based on the Penn plan of 1682, comprising a width of about eight blocks, extending north and south within the area between the Delaware and Schuylkill Rivers.

In a recently compiled "History of Pennsylvania", I have discovered what is believed to be an account against William Penn for the work done on this plan. The items are as follows:

"For taking courses and the soundings of the Delaware, seven weeks with Captain Markham, £10.

"To victuals and drink put on board the shallop at different times, £3.

"To my attendance at first commission William Hague, Nat Allen, and John Beazor, no charge."

(Even then, it seems, engineers were prone to give public advice free of charge.)

"To my taking courses of Schuylkill, etc., for sounding and placing Philadelphia upon Schuylkill, etc., £6.

"To lodging Captain Markham and William Hague in my house with diet and liquor for treats, £7.

"Total, including liquors, £26."

In 1791 there was produced, for a small section of what is now the capital of this country, a plan patterned to some extent after the Paris idea of the Seventeenth and Eighteenth Centuries as further developed by Baron Haussmann, under Napoleon III.

It is asserted that the early conception of the rectangular plan became an imprint on the municipal landscape of the United States, due largely to the authorized system of Government land surveys, adopted in 1785 at the suggestion of Thomas Jefferson. Whatever may have been the actuating cause, it is true that, with few exceptions until recent times, the plans of American cities have followed this line of least resistance as far as surveying is concerned. Consideration of topography played but little part.

One notable exception is that of Buffalo, N. Y., laid out by Joseph Ellicott, the younger brother of one Andrew Ellicott, who was the first Surveyor-General of the United States and who completed the plan of the City of Washington on the basis of the designs of Major L'Enfant. Detroit, Mich., in 1805, had a plan fashioned somewhat on the L'Enfant idea of diagonal thoroughfares. With these exceptions, until the time of the European and English Garden City movement, there was little variety in the planning of American cities. Kansas City, Mo., presents a case in which imposition of the rectangular plan has recently been greatly modified and delicately improved by the intelligent use of boulevards, parkways, and parks, and which has stimulated in the outskirts residential subdivisions of a variety and attractive character fitting the topography.

Another notable exception, artistic in its conception at the time, was the layout of the Centennial Exposition held in Philadelphia fifty years ago, followed by the more modern conception of Burnham for the World's Fair in Chicago, Ill., in 1902; and, later, at Buffalo, St. Louis, and San Francisco, Calif. These artistic and attractive developments made their impression on the work of the architect, the builder, and the engineer in the layout of municipalities and subdivisions. Notable examples were brought forward at the time of the World War, during which many towns and houses for workers were built after the highest ideals of usefulness, economy, and attractiveness of layout.

The fundamental basis of good city planning is the groundwork of the city engineer. If he is true to his calling he is always a city planner in embryo, because he must think ahead in planning public works and utilities. His understanding and appreciation of many of the elements of its attractiveness and his contribution to efficiency and economy have received much impetus during the past quarter of a century and it is only reasonable that special groups of engineers should organize themselves as city planners.

City planning has become an important issue in many large cities. It is to be regretted that its practice has been so long delayed. The recent problem of traffic relief has thrust itself suddenly on the engineer planner, who has been dismayed at the appalling cost of the reconstruction necessary to meet present-day traffic conditions.

Tunneling

Tunnels are built in the interest of transportation. The first tunnel in the United States, completed in 1821, was a part of a canal. The second, finished shortly afterward, extended the operations of a coal mine. Tunnels have been used widely for purposes of water supply and even sanitation; but their extensive use is found in connection with railroads. Just as the bridge may "level up" lines across a valley, so the tunnel can "level down" lines at summits.

In recent years highway traffic has become of such economic importance that the vehicular tunnel has laid claim to the services of the engineer. Just as it has always been necessary to carry railways and highways by bridges over waterways to preserve the rights of navigation, so, in these later years, the tunnel has been brought into service to relieve the streets of such traffic as may well be handled underground. It is within the past twenty-five years only that the engineer has been giving attention to the subaqueous tunnel that may provide facilities not readily supplied by a ferry or bridge.

Tunneling has been a relatively expensive operation. This nation began its existence one hundred and fifty years ago with much hope and courage, but with a lean purse. Even the need for transportation had to be created. When the necessity arrived, it had to be met with plans that felt the pinch of economy, for the Appalachian Range loomed as a barrier against any extension of territory to the west. This obstacle could for a time be avoided by way of the Great Lakes, which idea led to an all-water route across New York State to Lake Erie and from Lake Erie to the Ohio River.

The next plan that found favor was the shortening of this long journey to the Middle West by way of the Pennsylvania Canal and the incline planes of the Allegheny Portage Railroad. The heads of the inclines on the two sides of the mountain were connected through the first railroad tunnel constructed on this Continent, which was completed in 1833. Through this tunnel the eastbound traffic of the Pennsylvania Railroad still passes. Since the completion of this first tunnel the Appalachian Range has held no terrors for the engineer. He has pierced its sides and foot-hills many times in the interest of the railroads.

An epoch in the history of American tunnels was the completion, fifty years ago, of the Hoosac Tunnel, which extends for 4¾ miles under the Hoosac Mountains in Western Massachusetts. This enterprise met with many discouraging delays during the twenty years between its commencement and its completion. Since that experience many railroad tunnels have been built of different lengths, with the Moffat Tunnel in Colorado, 6½ miles long, as the most ambitious example. As a rule, all such tunnels were driven through rock. There came a day, however, when the engineer was compelled to work through soft ground above open water, and, later, to drive subaqueous tunnels through all classes of material.

One of the earliest of the important subaqueous tunnels is the pair of tubes for the Hudson and Manhattan Railroad (New York, N. Y.). It outranks the Hoosac Tunnel as to discouragements and delays, there being an interval of twenty-six years between its start and its finish. The difficulties with these two noted tunnels were sources of great instruction to the engineer. The lessons there learned have made the later problems in tunnel work extremely simple.

The so-called tunnels for subways under city streets have been constructed, for the most part, in open cut, the finished form of the subway being rectangular, of structural steel, encased in concrete. The skill of the engineer in this class of work is directed mainly to maintaining traffic at the street surface, protecting the foundations of existing structures at the building line, and permitting all sub-surface utilities to be operated continuously.

There are certain inventions or practices that have introduced epochs in tunnel driving, and that have combined to bring the art to its present degree of perfection. The rock drill was an American invention of 1849. The compressed air drill, in connection with high-grade explosives, spelled success for the Hoosac Tunnel in the early Seventies. The air-lock made the use of compressed air in subaqueous work possible. The adoption of the shield method for driving tunnels, first used in 1869, has eliminated the hazards of driving through soft materials. In its first use it was driven forward by hydraulic jacks, but, nowadays, compressed air under which tunnel work is usually carried on, more often is used as the driving force.

Water Supply

The first water-works plant in the United States was built in 1652. The supply was by gravity from springs. Pumping machinery was used for the first time in 1754. The pump was of *Lignum vitæ*, and the pipe to the reser-

voir was of hemlock logs. In 1862 this pump was replaced by others made of iron.

Steam was first used for pumping in 1800. The engines were constructed largely of wood, even the boiler being partly of this material. Wooden pipes were used entirely, and not until 1804 were cast-iron pipes introduced.

In 1800 the total number of water-works plants in the United States was 16, all, of course, in the larger cities. In the next fifty years this number increased to 83. In 1875 there were 422; by 1900, there were about 3 500.

By 1866, American water-works engineers had begun to take an interest in foreign practice with respect to filtration. The first slow sand filter plant in America was built in 1871, but it was not until 1880 that any considerable attention was paid in the United States to the purification of water supplies. Fanning's "Treatise on Hydraulic and Water Supply Engineering" published in 1884 (the first American work on the subject) contained only nine pages descriptive of filtering processes. By 1900, however, 20 cities in the United States were supplied with water that had been purified by the English or slow sand method of filtration, while 149 cities and towns were using mechanical filters, many being of the pressure type. Soon after 1900 concrete began to replace wood and steel tanks for mechanical filters and the majority of such filters built since that time have been of the gravity type. Recent practice favors the mechanical type and many of the existing slow sand filter plants are being rebuilt with "pre-filters" of that type.

To-day, more than 10 000 cities and towns in the United States have public water supplies. This tremendous growth has been the result of the requirements both for supplies of pure water for domestic and other uses and for fire protection. It has been made possible by the advent of cast-iron pipe, the invention of improved pumping machinery, and the development of advanced methods of purification. One of the most noticeable changes in the art has been that which has taken place in pumping machinery within the last fifteen or twenty years. Increased economy had been attained by the design of great direct-acting, compound, and triple-expansion pumping engines. Within recent years, however, motor-driven turbine or centrifugal pumps have replaced the direct-acting engines on account of their lower first cost and economy in operation.

Another development of the last thirty years has been the quite general introduction of the meter system. With the adoption of meters has come a change in the method of charging for water from the former flat, or fixture, rate to schedules based on the quantity of water used. This method reduces waste, promotes economies in operation, and tends to maintain expenditures for the pumping and filtering plants at a minimum.

Sewerage and Sewage Disposal

The development of sewerage systems and methods of sewage disposal has been coincident with the growth of cities.

The first application of engineering skill to the design of sewers was about 1857-58. Empirical formulas, however, were used as a basis of design until, in 1889, the rational method was advocated. The combined system of sewers

was in general use up to 1880 when the first systems of separate sewers were constructed.

The earliest sewage pumping stations were put into service in 1884 and 1885. As the discharge of raw sewage into the streams increased, their condition became intolerable. In 1872, investigations relative to stream pollution were begun by the State Board of Health of Massachusetts. In 1887, the Lawrence Experiment Station of the Massachusetts State Board of Health was established. The researches of this Station in connection with the fundamental biological conditions underlying processes of sewage treatment constitute an unparalleled contribution to the art of sewage disposal.

Among the first methods of sewage treatment in the United States were attempts at sub-surface irrigation and broad irrigation. Broad irrigation was soon replaced, particularly in New England, by intermittent sand filtration. Chemical precipitation followed in the latter Eighties. Contact beds for the oxidation of organic solids in sewage were introduced into America during the next decade. Trickling filters have largely superseded contact beds. The first municipal filter of this type was put into service in 1908, and the fine screening of sewage was introduced into this country in the same year.

The earliest American installation of Imhoff tanks was made in 1911, and the first attempt at separate sludge digestion on a large scale, shortly afterward.

The most recent development of importance in sewage treatment has been in the activated sludge process, which is the outcome of experiments on the aeration of sewage by numerous American workers, particularly at the Lawrence Experiment Station in 1912. The first activated sludge plant was built in this country in 1916, and several large plants have been constructed since that date.

Disinfection of sewage was practised as early as 1892 by the use of chlorine produced electrolytically. The efficiency of hypochlorite of lime was demonstrated later. At present, liquid chlorine is more widely used.

Structural Engineering

One-half of the one hundred and fifty years with which we are dealing had passed before metal was used as a building material. In this earlier period outside and intermediate walls and interior rows of wooden posts bearing longitudinal timbers, all supporting the floor joists, carried the weight of the building and its contents to the foundation walls, which were of the continuous type. With this character of construction, buildings of more than five stories were seldom built, because such structures were not readily accessible in their upper floors, and, in addition, required walls and foundations of unreasonable thicknesses, thus adding greatly to the dead weight of the building. The increased fire and wind hazards were also important factors.

About 1850 cast iron came into use in the form of columns and beams, and later it was used very extensively in the fronts of business houses for ornamental effect. A large five-story building covering a city block in New York with its four faces of cast iron, built in 1859, is still in use. The frame-

work of the Crystal Palace, located on the present site of Bryant Park, New York, and built in 1853, was entirely of cast and wrought iron.

Iron beams were first rolled in 1854. It was probably as late as 1870, or about fifty-five years ago, that beams, channels, angles, and tees were available. About 1884 the rolling of shapes changed from iron to steel, and within ten years but little iron was rolled into structural shapes.

With the advent of the rolled beam efforts were made to reduce the risks from fire in large buildings by using these shapes as floor-beams between which were sprung brick segmental arches. This practice prevailed generally until the Chicago fire in 1871, during which incident the disastrous effects of fire on the beams was demonstrated. Thereafter resort was had to other practices that involved various forms of hollow tile, and when arches between metal floor-beams were of this material, the beams themselves were encased in hollow tile forms. Intermediate arches of light weight, such as of cinder concrete, also have been popular. Of recent years the reinforced concrete slab floor has found great favor.

There came a time when it was recognized that land areas must be used more intensively in the business sections of cities. In the vertical expansion of buildings to meet this necessity the important problem was to provide easy access to the higher floors. It was sixty years ago that the first suspended type of elevator was installed. The first elevators were steam-operated, the suspending cable being wound around a drum. The hydraulic plunger type followed. Now, the electric elevator has largely superseded the earlier types.

Thanks to the introduction of structural shapes and the elevator, buildings began to creep skyward. By 1885, ten-story buildings were being built. In 1887, skeleton construction was established by its use in all but party walls. In 1891, or just thirty-five years ago, the use of weight-bearing walls for high buildings ceased, and the full skeleton type of construction had been adopted. At that time a twenty-story height was attained. The practice of supporting the walls of each floor independently of the others and upon the skeleton structure permitted the discarding of continuous foundations and the adoption of isolated footings as supports for the posts.

Buildings of forty stories in height are becoming quite common, and there are a few examples of even fifty stories. The United States was somewhat behind England in the use of metal columns and beams in building construction, but, once the wrought-iron shapes were available for this purpose, the American engineer moved swiftly to the skyscraper, which has proven to be a most wonderful structure, in that it conserves space horizontally, provides as nearly as possible fire and storm-proof quarters, and affords better air and natural light, while its floor spaces are adjustable to any combination of partitions. From the first rolled beam to the all-skeleton building was but thirty-seven years.

Within the past twenty years great advances have been made in the quality and quantity of Portland cement. The use of this material in connection with reinforcing bars (a new shape in steel) in casting beams and posts in place has brought reinforced concrete into competition with steel in skeleton constructed buildings.

Irrigation

Probably no phase of the Engineering Profession is more tinged with romance than the work of the irrigation engineer. The creation of fertile fields, where once the sage and cactus suffered a shriveled existence, lends itself too readily to fanciful dreams to escape glorification by authors and scenario writers.

Irrigation dates back several thousands of years before the time of Christ, when the fields of Egypt, once desert lands, were transformed into garden spots by primeval engineers who constructed hydraulic works to control the periodic overflow of the Nile for its deposit of rich alluvial mud. Its ancient and honorable status is also attested by the fact that practically the first Scriptural reference to agriculture relates to irrigation in the Valley of the Euphrates, for in Genesis, 2:10, we find this statement:

"And a river went out of Eden to water the garden; and from thence it was parted, and became into four heads."

Irrigation continues in practice in the Far East, and, although it is still, to some extent, using devices similar to those employed centuries ago, some of the world's major works, and some of the most advanced practices in irrigation, are to be found in India and Egypt.

Excepting only the primitive efforts of the Indians along the Southwest border, irrigation was not practiced on any scale in the United States until 1847. The plains of the Middle West were so readily accessible for agriculture that it was unnecessary to resort to the more expensive development of farm irrigation. The hopelessness of attempting agriculture by ordinary methods, in that region of restricted rainfall which they then occupied, led the Mormons to initiate irrigation. This movement has since been developed to such an extent that there are to-day in the United States more than 63 000 individual irrigation enterprises. Early irrigation works, while simple when compared with more recent practice, demonstrated the possibilities of using the once worthless desert land for raising crops.

Irrigation was limited to the smaller and less expensive developments until 1902, when the plan was inaugurated of using Government funds, derived from the sale of public lands, for the construction of large projects, the funds thus expended to be repaid to the Government by the settlers on the land reclaimed. This work has been prosecuted under the direction of the United States Reclamation Service, and the great dams and reservoirs built by this organization are outstanding monuments to modern engineering skill. To date, this agency has expended about $165 000 000 on construction works alone and has reclaimed nearly 2 000 000 acres of land. The cropped area on Federal projects for the year 1925 was 1 216 610 acres, and the yield that year was valued at $66 488 500, or $54.65 per acre.

The Eleventh Census of the United States, compiled in 1889, shows 3 564 416 acres as being irrigated. In the following ten years, this figure was more than doubled to a total of 7 263 813 acres in 1899. In 1919, there were 231 541 irrigated farms with a total of 19 191 716 acres being irrigated, this being nearly 2% of all the farm land, and 3.8% of the improved farm area,

of the United States. The total capital invested in irrigation enterprises in the United States, as reported by the last Federal Census (1920), amounted to $697 657 328, and the total value of irrigated crops for the census year approximated $800 000 000.

The economic value of irrigation in respect to Western agricultural and industrial expansion, and the important relationship that this expansion bears to National development and security, in both its economic and military aspects, can hardly be over-estimated.

The Civil Engineer deals with quantity, time, cause, and effect. His problems involve dimensions, form, and motion. He creates to-day and destroys to-morrow, but out of the destruction new forms arise of improved design and greater utility. Experience may be a guide for his future policy, but he cannot rest on repetitions of methods that may have brought him fame. He must be alert to the ever changing conditions about him, recognize the problems they create, and apply wisdom and ingenuity to their solution.

He accepts physical laws as immutable. In the past one hundred and fifty years he has developed no new ones, but he has subjected the familiar ones to a more searching analysis. New forms of fuel have been found; the steam engine has been developed; electricity born; and the key to distribution of power found. Less drudgery, better health, and more leisure—these are the realities of the American life to-day to which the Civil Engineer has contributed.

STRESSES IN HELICALLY REINFORCED CONCRETE COLUMNS

Discussion*

By A. W. Zesiger, M. Am. Soc. C. E., and
E. J. Affeldt, Assoc. M. Am. Soc. C. E.†

A. W. Zesiger,‡ M. Am. Soc. C. E. and E. J. Affeldt,§ Assoc. M. Am. Soc. C. E. (by letter).‖—The desirability of data regarding the deformations of columns, with and without helical reinforcement, under working loads, is mentioned in Mr. Thompson's discussion.¶ Information of this nature for cylinders and plain concrete columns under compression is fairly abundant. In such data the modulus of elasticity is given, and there is really much better information regarding the behavior of concrete under loads up to those that will produce a shearing failure of plain concrete than for loads in excess of these. The writers, however, are more concerned with the action that takes place after the shearing strength of plain concrete has been reached, in order to ascertain wherein helical reinforcement exerts such an influence on the ultimate strength of a column thus reinforced. As stated in the paper,** the constants used in a working formula should always be predicated upon the properties of the material at failure. In this manner only is it possible to determine any given factor of safety, since, as is well known, the values of n, etc., do not by any means vary in direct ratio to the load.

The effect produced in the diameter of the helix by the shortening of the column under load, and its consequent lateral expansion, is far less than that assumed by Mr. Thompson. It is really so small that it may be considered negligible, in all cases, as the following extreme example will show. In addition to the notation previously used,††

Let L = the length of one unstressed coil of the helix.

s = the pitch of the unstressed helix.

f = the compressive stress in the concrete.

s_1 = the pitch of the helix when stressed.

D_1 = the diameter of the helix when the pitch becomes equal to s_1.

D_2 = the diameter of the concrete core due to expansion.

* Discussion on the paper by A. W. Zesiger, M. Am. Soc. C. E., and E. J. Affeldt, Assoc. M. Am. Soc. C. E., continued from August, 1926, *Proceedings*.

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‖ Received by the Secretary, July 24, 1926.

¶ *Proceedings*, Am. Soc. C. E., April, 1926, Papers and Discussions, p. 733.

** *Loc. cit.*, January, 1926, Papers and Discussions, p. 3.

†† *Loc. cit.*, p. 9, *et seq.*

Then,

$$D = \frac{1}{\pi}\sqrt{L^2 - s^2}$$

$$s_1 = s\left(1 - \frac{f}{E_c}\right)$$

$$D_1 = \frac{1}{\pi}\sqrt{L^2 - s^2\left(1 - \frac{f}{E_c}\right)^2}$$

$$D_2 = D\left(1 + \frac{f}{m\,E_c}\right)$$

$$\frac{D_1}{D} = \frac{1}{\pi}\sqrt{\frac{L^2 - s^2\left(1 - \frac{f}{E_c}\right)^2}{(L^2 - s^2)}}$$

$$\frac{D_2}{D} = 1 + \frac{f}{m\,E_c}$$

Assume that, $D = 6$ in.; $s = 2$ in.; $n = 27$; $E_c = \frac{30\,000\,000}{27} = 1\,110\,000$; $m = 2.5$; and $f = 3\,704$ lb. per sq. in. Then, $\frac{D_1}{D} = 1.0000316$; $\frac{D_2}{D} = 1.001145$; and $\frac{D_2 - D}{D_1 - D} = \frac{0.001145}{0.0000316} = 36.2$, or the increase in diameter of the helix due to shortening of the column is less than one-thirty-fifth of that due to lateral expansion of the concrete for this extreme case; for columns of larger diameters it is proportionately less.

The column formula deduced by Mr. Thompson is quite ingenious, but is predicated on the same defective assumption as that of Euler, namely, that an ideal column will deflect without any force causing it to do so, whereas an ideal column with the load axially applied should not deflect. His demonstration would seem to indicate, however, that flexure does not cause failure of a concrete column for values of $\frac{l}{r}$ less than 100, and the writers, therefore, would appear to be conservative in suggesting a ratio of $\frac{l}{r}$ not greater than 80 as the safe limit.

Referring to Professor Eddy's discussion* the writers are uncertain whether he takes exception to their original use of the word "incipient" or implies that there is no friction between the particles of a concrete column until rupture has occurred. In using the word "incipient" the writers had in mind a relation between the particles of a column such that motion was on the very verge of commencement, but had not yet started. This is probably an incorrect interpretation and the word has, therefore, been omitted.

If, however, Professor Eddy means to imply that friction cannot exist between the particles of a column until rupture has occurred, and that friction and shear, therefore, can not occur simultaneously, the writers are

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* *Proceedings*, Am. Soc. C. E., May, 1926, Papers and Discussions, p. 1006.

unable to agree with him and feel confident that the consensus of engineering opinion will not support his view. If the latter interpretation of his discussion is correct it would be interesting to know how Professor Eddy reconciles his view with the fact that concrete columns do not shear along a plane having an inclination of 45° with the horizontal, since the shearing stresses would be a maximum and the shearing resistance a minimum along such a plane.

The writers feel indebted to Mr. Harder* for his interest in their paper and his comparison of their treatment of the subject with that of Rudolf Saliger. The practicability of theoretical reasoning, when applied to a material such as concrete, is, of course, a matter of opinion and Mr. Harder has a perfect right to his views. A study of literature pertinent to the subject of concrete construction would seem to indicate, however, that by far the majority of the design formulas now in use are based on theoretical reasoning.

Like Saliger, the writers originally made the same error in Equation (17)† and omitted the term, $\frac{h}{m\,E_c}$, but later discovered the mistake. They cannot agree with Mr. Harder's contention that Equation (17) does not obtain unless the helix is wound on the concrete core under tension. It is undoubtedly true that the concrete core in setting has a slight tendency to shrink away from the helix, but this can be counterbalanced by assigning proper values to the constants. On the other hand, if the helix could be wound on the concrete core under tension, the lateral pressure against the concrete would thereby be increased unless the tension in the helix could be adjusted so as to compensate exactly for the shrinkage of the core, which is, of course, practically impossible, and Equation (17) would no longer apply. Furthermore, the writers cannot agree with Mr. Harder that the omission of the term, $\frac{h}{m\,E_c}$, will simplify in any way the final result as given in Equation (21)† and especially in the working Equation (28*a*).‡ About the only difference between Equation (21) and Mr. Harder's equation,§

$$v = \frac{f'_c}{1 - \tan^2\left(45^\circ + \frac{1}{2}\,\phi\right)\frac{n\,p'}{m\,(2 + n\,p')}}$$

is that the former is theoretically more nearly correct.

Mr. Harder's attempt to show that his formula,‖

$$v = f'_c\left(1 + \frac{m\,n}{2}\,p'\right)$$

is correctly deduced from the writers' Equation (21), is hardly successful.

* *Proceedings*, Am. Soc. C. E., May, 1926, Papers and Discussions, p. 1006.

† *Loc. cit.*, January, 1926, Papers and Discussions, p. 12.

‡ *Loc. cit.*, p. 14.

§ *Loc. cit.*, May, 1926, Papers and Discussions, p. 1010.

‖ *Loc. cit.*, p. 1009.

Although his demonstration is an ingenious example of algebraic manipulation, he seems to have lost sight of the basic facts on which the theoretical equation is predicted. When $p' = 0$, the equation, $f_c' = \frac{m}{n} f_s$, is correct, but for that value of p' only. In other words, f_c' is a constant for any given concrete, whereas f_s varies as p'. As p', and consequently the quantity of steel, increases, the lateral restraint of the column becomes greater and the vertical pressure necessary to produce a lateral deformation that will cause rupture of the concrete is proportionately increased, resulting in a greater longitudinal and lateral deformation and, consequently, in an increased value of f_s. The equation, $f_c' = \frac{m}{n} f_s$, is, therefore, incorrect for any values of p' other than 0. It follows that the equation,

$$v = f_c' + \frac{m}{2}\, p'\, f_s$$

is likewise incorrect, and the equation,

$$v = f_c' \left(1 + n\,p + \frac{m\,n}{2}\, p'\right)$$

therefore, cannot be predicated on theory.

The coefficient of p' in the third term of the right-hand member of Mr. Harder's equation increases as m increases, whereas it should theoretically decrease. Consequently, the German design formula,

$$v = f_c\, (1 + 15\, p + 45\, p')$$

cannot be founded on theory, but must be purely empirical. Inasmuch as the equation,

$$v = f_c' + \frac{m}{2}\, p'\, f_s$$

is incorrect, Mr. Harder's conclusion that

$$m = \tan^2 \left(45^\circ + \frac{1}{2}\, \phi\right)$$

must likewise be considered erroneous.

Referring to that part of Mr. Harder's discussion wherein he states that, "if one were interested in the history of theoretical derivations, the statement that, until recently, writers have neglected friction in the concrete, will need some amplification", the writers are well aware of the fact that friction in the concrete had been considered prior to the appearance of their paper. In fact, they explicitly stated* that Navier has given the true analysis of granular substances in compression. The late J. B. Johnson, M. Am. Soc. C. E., and a few others, including Saliger, also have taken friction into account, but the great majority of engineers seem to have neglected it even though they may have been aware of its existence.

In using Bach's tests on prisms Mr. Harder seems to confuse the strength of cubes with what the writers term the crushing strength of concrete, as

* *Proceedings*, Am. Soc. C. E., January, 1926, Papers and Discussions, p. 5.

even cubes, when tested to destruction, will fail partly by shearing. Although the writers have never made or witnessed tests wherein the ratio of height to breadth was 0.5, it would appear to them that, in such a case, the ratio of shearing to crushing failure would be very small. A ratio of 0.85 to 1.41, although still somewhat too low, would represent a more nearly correct relation between shearing and crushing strength; that is,

$$v_c = \frac{1.41}{0.85} f_c' = 1.659\, f_c'$$

From the foregoing it should be obvious that Mr. Harder's determination of *m* from Bach's tests is in error.

The writers were well aware of the complication of stresses in cylinders subjected to axial stress, as described by Professor Griffith*. It was, however, and still is, their belief that the principal stresses causing failure in a short, helically reinforced concrete column are so-called shear and crushing. In using the expression "so-called" the writers have in mind that every failure, whatever its nature, or however it may be designated, is in reality some type of tension failure. When the cohesion or bond, by whatever term it may be designated, between the particles of a substance is ruptured, failure of the material takes place. Whether the particles are pulled apart, pushed apart, or twisted apart, the fact remains that they are separated. For the sake of convenience, and as a sort of mask for the prevailing ignorance concerning the phenomena attendant on failure of the material, it is customary to speak of shearing failures, tension failures, etc. It is the writers' belief, however, that the ultimate cause of all these types of failure is some kind of tension. In so-called crushing the principal factor tending to produce disintegration of the material seems to be an induced tension at right angles to the line of action of the principal stress that is, tension along the periphery of the material.

As Professor Griffith has so clearly stated, the majority of tests on concrete columns thus far reported are sadly lacking in the refinements essential to the attainment of precise data, and there is urgent need of further research along the proper scientific lines.

In his discussion Mr. Tucker† states that certain equations are "not valid", that many of the conclusions are erroneous, etc. The writers frankly admit that they are unable to follow his criticisms. In view of the fact that Mr. Tucker so often gives an interpretation to their language and mathematics that is at utter variance with what is stated, they are constrained to believe that the majority of his criticisms are based on a misinterpretation of the paper. Had he offered any proof of his various assertions as to the non-validity of the writers' mathematics, etc., they would gladly have attempted to answer him.

In reply to the various points raised in Mr. Feld's discussion‡ the writers wish to state that:

(*a*) It is true that their mathematics assumes a uniform shearing plane. They are aware, of course, of the fact that no granular mass will shear along

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1148.
† *Loc. cit.*, p. 1153.
‡ *Loc. cit.*, p. 1160.

a single plane. The plane assumed by them is merely the average of all planes.

(*b*) By the expression "little effect in preventing crushing", etc., the writers do not mean to imply that helical reinforcement has no effect. Its effectiveness in preventing a crushing failure is, however, so much less than in the prevention of a shearing failure that it does not seem economical to use steel for the former purpose. This is indicated by the graphical representation of Equation (23)* in Fig. 4.†

(*c*) There do not seem to be sufficient data available to indicate whether, if enough steel is used to preclude a shearing failure, crushing of the concrete will occur at or below the elastic limit of the steel.

Mr. Feld is correct in assuming that r is the radius of gyration of the column. The ratio of length to minimum width of column is undoubtedly a more simple quantity, but the writers are unable to understand wherein the use of such a relation would tend to minimize possible errors. Inasmuch as, for a constant width (or diameter) of column, r varies with the amount of reinforcement used and, therefore, takes it into account, whereas any relation between the length and minimum width (or diameter) of column ignores it, the use of the ratio of length to least radius of gyration would seem preferable.

Referring to that part of the writers' paper‡ wherein it is demonstrated that the extreme fiber stress that will shear out a wedge must be about 1.75 times the average stress that will cause the column to shear across its entire cross-section, they did not state whether this extreme fiber stress equalled the crushing strength of the concrete. There are few data from which to determine the crushing strength of concrete. It would seem, however, that such strength is approximately equal to 1.75 times the stress that will cause shearing across the entire column cross-section. This, of course, applies only to plain concrete columns. Mr. Feld's statement that the authors' equations "give the same possibility of failure for a given load placed axially or with an eccentricity of one-eighth the diameter," is not correct even for plain concrete. The flexure that an eccentrically applied load will cause in a column of any appreciable length will produce an additional eccentricity resulting in an extreme fiber stress of more than 1.75 times the average. This action is even more noticeable in reinforced (especially helically reinforced) concrete columns in which the average stress approaches the crushing strength of the concrete. Even in the case of columns reinforced only with longitudinal rods tied together at intervals with bands or wire the steel resists a shearing failure to a greater or lesser extent and, therefore, increases the average stress in the concrete necessary to produce a shearing failure.

The writers were aware of the fact that shrinkage of the concrete in setting will affect the value of the helix, but believed that the constant selected for m in their working equation would tend to minimize this error.

* *Proceedings*, Am. Soc. C. E., January, 1926, Papers and Discussions, p. 12.

† *Loc. cit.*, p. 18.

‡ *Loc. cit.*, p. 7.

VIRTUAL WORK: A RESTATEMENT

Discussion*

BY MESSRS. J. CHARLES RATHBUN AND HARDY CROSS.†

J. CHARLES RATHBUN,‡ M. AM. SOC. C. E. (by letter).§—Professor Cross has pointed out the application of the theory of virtual work when the fibers of the structure are in tension or compression, and the manner of computing deflections in a given direction by assuming a hypothetical force acting in that direction. He also mentions the method of computing rotation based on tension and compression in the fibers.

If, in addition, shear in the fibers is considered, the torsional deformation (as in a shaft) will also apply, as well as the deformation due to simple shear. Thus, in the cantilever beam of Fig. 10, to obtain the full deformation in the direction of the force at the point, *P*, not only is the deformation due to the direct thrust in the direction of the *X*-axis and the moments about the *Z* and *Y*-axes to be taken, but also the shear in the direction of each of these axes and the torque about the *X*-axis. As a rule these shears may be neglected, while the torsion does not often occur in engineering design. However, when it does, it may become an important element in the deformation.

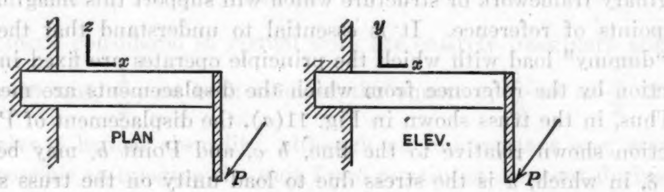

FIG. 10.

Each one of these six elements of the deflection can be obtained by a separate computation by virtual work.

HARDY CROSS,‖ M. AM. SOC. C. E. (by letter).¶—The writer has read with interest the discussion by Mr. Doerfling,** but is unable to accept his conceptions either of virtual work or of the laws of statics. Mr. Doerfling states††

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* Discussion of the paper by Hardy Cross, M. Am. Soc. C. E., continued from September, 1926, *Proceedings*.

† Author's closure.

‡ Head of Dept., and Prof., Civ. Eng., South Dakota State School of Mines, Rapid City, S. Dak.

§ Received by the Secretary, August 20, 1926.

‖ Prof. of Structural Eng., Univ. of Illinois, Urbana, Ill.

¶ Received by the Secretary, October 29, 1926.

** *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1480.

†† *Loc. cit.*, p. 1481.

that "during the transition from one state of equilibrium to the other, each panel point moves with all its forces in equilibrium", a view of the phenomenon of motion which is also stated algebraically. This, if correct, would be an important addition to the laws of physics.

The equality assumed by Mr. Doerfling* that $S_1 = P_1$, is simply an identity. The quantities, $\Delta\, l_2$ and d_2, are by definition the movement of a point in a given direction (the direction of P_2 and S_2). Hence, $S_1\, \Delta\, l_2 = P_1\, d_2$. Mr. Doerfling is correct in stating that this is not an equation of real work; the terms, "hypothetical equation" and "imaginary equation", are new to the writer and he is unable to interpret them.

Apparently, Mr. Doerfling is trying to deduce displacement relations from considerations of the true internal work of structures. Of course, such a method, if logically applied to elastic structures, leads to equations identical with those reached by virtual work; and it may even appear to lead to the same results if followed illogically, but with determination and purpose. Considerations of real work, however, are not used in the method of virtual work, a method which Mr. Doerfling has failed entirely to grasp, since he states that the inclusion of imaginary reactions or hypothetical stresses is unnecessary and confuses his mind.

The method of virtual work leads to the equation, $\Delta = \Sigma\, u\, \delta$, in which, Δ is the unknown external movement resulting from the internal distortions, δ. The meaning of the term, u, however, is not always made clear nor are the limits of the summation. Perhaps the most comprehensive statement is that the u-values are the imaginary internal resistances to distortion (internal stresses) produced by an imaginary unit load, coincident with Δ, in any imaginary framework or structure which will support this imaginary load from the points of reference. It is essential to understand that the reactions to the "dummy" load with which the principle operates are fixed in position and direction by the reference from which the displacements are measured.

Thus, in the truss shown in Fig. 11(a), the displacement of Point a, in the direction shown relative to the line, $b\ c$, and Point b, may be found from $\Sigma\, u\, \delta$, in which, u is the stress due to load unity on the truss shown in Fig. 11(b). However, any framework that is made up of any or all of the bars of the original truss and that would support a load at a from Points b and c, might as well have been chosen. A framework as shown in Fig. 11(c) might even be used provided the value of δ were known for the false bar shown by dashes.

Also, in the beam shown in Fig. 12(a), in finding the displacement of a with reference to b and c, u may be taken as the moments along the chord, bc, due to a unit load at a and then the integration, $\int u\, \phi$, may be performed; or u may be taken as the stress in the truss shown in Fig. 12(b), or in any such framework connecting a with b and c and $\Sigma\, u\, \delta$ may be applied to the imaginary bars.

The principle is entirely a geometrical tool derivable from the principle of the conservation of energy. It does not deal with, nor does it depend on,

* *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1482.

considerations of true internal work. It is not, therefore, dependent on the validity of Hooke's law, as Mr. Doerfling seems to think. That it is to be found almost exclusively in the literature of engineering results from the fact that only engineers find it especially convenient.

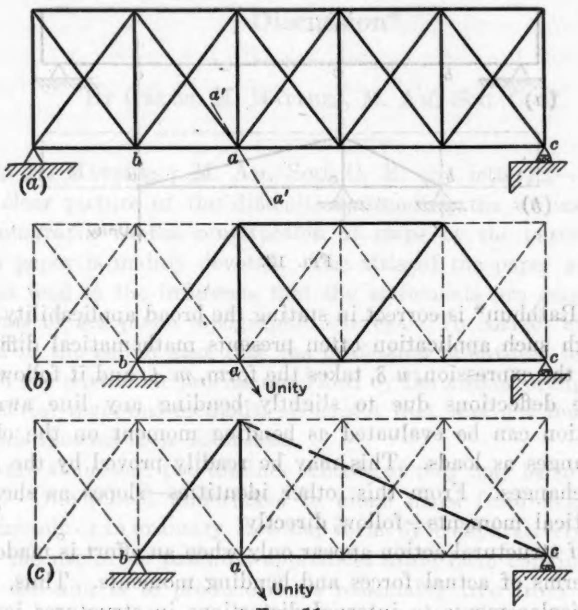

FIG. 11.

The phenomena considered in virtual work are entirely imaginary and the use of the coefficient, $\frac{1}{2}$, on both sides of the equation, $\frac{1}{2}\,\Delta = \frac{1}{2}\,\Sigma\, u\, \delta$, seems to involve a logical absurdity, although an unimportant one, since it is obviously possible to imagine the unit force remaining constant during the deformation and producing constant resistances in an inelastic structure.

The interchangeability of s with u and of m with m' in the expressions,

$\Delta = \Sigma\, \frac{s\, u\, l}{A\, E}$, or $\Delta = \int m\, m'\, \frac{d\, s}{E\, I}$, furnishes so simple and complete a proof of the reciprocal theorem that the writer is surprised that Mr. Doerfling considered it necessary to substitute a demonstration which is both tedious and incomplete. Mr. Doerfling begins his demonstration with a statically determinate structure and at some point not defined extends it to indeterminate structures. The limitation of stress-strain proportionality makes it inadvisable to consider the reciprocal theorem as a fundamental.

Illustration of the utility of the reciprocal theorem does not properly belong in this paper, as the writer specifically stated. The writer tried to condense the paper as much as possible; Professor MacLean* thinks that he

* *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1480.

condensed it too much. Probably Professor MacLean is right, but the writer ventures the opinion that engineering papers in general are too extensive and that the literature of indeterminate structures especially has suffered from elaboration.

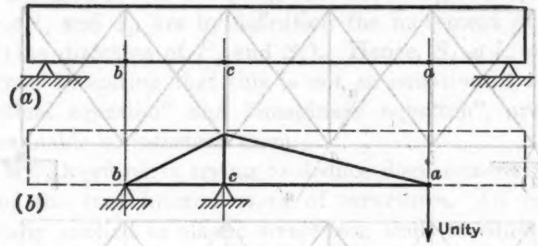

FIG. 12.

Professor Rathbun* is correct in stating the broad applicability of the principle, although such application often presents mathematical difficulty.

In beams, the expression, $u\ \delta$, takes the form, $m\ \phi$, and it follows as a corollary that the deflections due to slightly bending any line away from its original position can be evaluated as bending moment on the chord due to the angle changes as loads. This may be readily proved by the geometry of small angle changes. From this, other identities—slopes as shears, tangent offsets as statical moments—follow directly.

Theories of structural action appear only when an effort is made to evaluate δ and ϕ in terms of actual forces and bending moments. Thus, the relation of external displacements to internal distortions in structures is not subject to question; the values of these distortions in actual structures is subject to some question, but the significance of these distortions as indicating failure is subject to considerable dispute.

* See p. 1985.

AEROPLANE TOPOGRAPHIC SURVEYS

Discussion*

By Gerard H. Matthes, M. Am. Soc. C. E.

Gerard H. Matthes,† M. Am. Soc. C. E. (by letter).‡—The author has drawn a clear picture of the difficulties attending the utilization of vertical aerial photographs in the construction of maps by the particular process to which his paper is mainly devoted. The title of the paper and treatment of the subject lead to the inference that the statements are generally applicable to all forms of aeroplane topographic surveys. To correct any such impressions, and in the belief, also, that the profession is entitled to know the position which the particular process described by the author occupies with respect to other aerial mapping methods, an aspect that was not touched on by Mr. Bergen, this discussion is submitted.

There are at present two distinct schools, if they may be so styled, of aerial surveying. One school undertakes to make maps from aerial photographs, either in mosaic or in ordinary line map form, by comparatively simple methods involving the use of no patented appliances aside from copying and enlarging cameras. Its aim is to produce maps reasonably free from the various distortions described by the author, and based on the fewest possible number of control points established by ground survey. The general principles involved are so simple that—with proper training—any skillful engineer, surveyor, or draftsman can master them in a short time. In this school belong the aerial mapping activities of the U. S. Geological Survey, Topographic Branch, which have by this time reached an extensive state of development; the aerial mapping operations of the Dominion of Canada, the Topographic Survey of which now leads the countries of the world in the quantity production of aerial Government maps; the aerial surveys of the U. S. War and Navy Departments; and a host of lesser aerial surveys for corporations, municipalities, and private individuals. In most of these aerial surveys, scale errors and distortions such as are mentioned by the author and are normally inherent in aerial photographs, are either eliminated through correction by the radial orientation method,§ or reduced to negligible proportions through material reductions in the scale of the finished map.

In North America, to date, by far the majority of aerial maps has been produced by methods of this kind. Experience indicates that these methods

* Discussion on the paper by George T. Bergen, M. Am. Soc. C. E., continued from October, 1926, *Proceedings*.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, September 27, 1926.

§ For a description of the method of radial orientation, see *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), pp. 792–799.

have proved best adapted to surveys at the smaller scales, namely, those ranging from as small as 1:62 500 to as large as 1:5 000. Although adapted to larger scales than 1:5 000, these methods do not always effect material economies over mapping by ordinary ground surveys for the reason that at these larger scales the labor involved in correcting errors in scale, displacements, and tilt distortions, becomes considerable.

The other school of aerial surveying, in which belongs the process described by the author, makes use of highly precise optical-mechanical appliances and has for its chief characteristics: (*a*) The correcting of each individual photograph for tilt and scale errors by a process of re-photographing known as rectification or restitution (a term borrowed fom the French and widely used in Europe); and, (*b*) the determining of differences of elevation in the terrain through measurement of the parallax of photographic images common to overlapping exposures. The author has given the best description of the principles involved that has yet appeared in print. It is important to note that extensive ground control is required in order to insure accurate results, and that all exposures must be corrected for scale and tilt errors, even if the discrepancies involved are so small as to be inappreciable by the other methods of aerial mapping.

The process described by the author is the only one of its kind in actual commercial use in North America. It is not, however, the only process of its kind in existence. Half a dozen similar mapping methods are in operation in Europe, and as many more are in process of development in America and abroad. The total amount of inventive effort that has been and is being expended in this direction is surprisingly large in view of the high cost of the appliances and the rather limited commercial market for them. Thus far, Governmental mapping agencies have not elected to use equipment of this kind.

As a class, optical-mechanical mapping processes have proved to be well adapted to the making of maps on scales ranging from 1:10 000 to as large as 1:1 000, and have attracted considerable attention because of the ease with which contour lines may be produced mechanically, thus eliminating topographic surveying in the field. How well any of these processes (all of them patented) are adapted to producing maps on scales as small as 1:62 500 (the standard scale of the U. S. Geological Survey topographic quadrangles) remains to be seen. In Europe much experimental work has been done in this direction, but to date no small-scale surveys of large areas have been undertaken with apparatus of this kind.

The author stresses the refinements necessary for correcting the photographic data before they can be utilized in the map, and for determining differences of elevation. These refinements are characteristic of all optical-mechanical methods. However, it should not be imagined that the degree of precision to be maintained in this part of the work is necessarily present or reflected in the finished map. Its introduction is merely incident to dealing with parallax, the measurement of which involves exceedingly small quantities.

A numerical example will make this clear. Referring to Equation (1),* assume an ordinary case, namely, a distance, B, between centers of overlapping photographs of 2.5 in.; a camera altitude, H, of 4 000 ft.; and a scale of the photographic images of 1:4 800. Then, the difference in parallax, p, for a difference of elevation, h, of 10 ft., will be found to be 0.00043 in., a very small quantity indeed. Even for an object 100 ft. in height, under the conditions assumed, the difference in parallax would be only 0.0043 in.

Undoubtedly, the apparatus described by the author is fully capable of making measurements with this degree of refinement. The important point to observe is that after these precise parallax measurements have been made, the construction of the map—that is, the piecing together of the bits of information derived from the individual exposures—still remains to be attended to, as does also the correction of the contour lines and other features for displacements caused by perspective, and for scale errors caused by differences in elevation. In these respects, the process described by the author appears to be much the same as the other aerial mapping methods; and it is here that the real skill in producing an undistorted map from a multitude of exposures is required. It is here, also, that frequently much of the extreme precision involved in rectification and contour determination is lost again; for, as in other mapping processes, this part of the work is a hand-and-eye job subject to all the chances for error resulting from varying degrees of judgment, skill, and "pep" of the individual who transfers the lines and symbols from the photographs to the map and fits them to the ground control.

The author dismisses this part of the map-making process in less than two hundred words under the head of "Plotting and Tracing",† leaving the reader with an incomplete picture of how all this material that has been worked over with so much painstaking care finally finds its way into its appointed place in the map. It is highly desirable for the author to shed further light on this phase of the subject, which seems the more important since in the preparation of maps of all kinds the most prolific sources of error occur in plotting the detailed data, whatever their origin and accuracy may be, in their proper position. In some of the European optical-mechanical processes all map features, inclusive of contour lines, are drawn mechanically on the map by means of a pantographic attachment forming part of the stereoscopic apparatus, thereby eliminating, very largely, errors in drafting.

Owing to the extreme delicacy with which parallax measurements must be made, errors in the position of photographic images, however minute, due to film and lens distortions become sources of material error. As pointed out by the author, glass plates must be used instead of film, but his statement‡ to the effect that film is "inappropriate for the purposes of aerial mapping", is entirely too broad, being applicable only to aerial mapping by optical-mechanical processes involving parallax measurements.

Film has been and is being used much more extensively than plates in aerial work. Most of the larger aerial mapping agencies in the world, includ-

* *Proceedings*, Am. Soc. C. E., March, 1926, Papers and Discussions, p. 374.

† *Loc. cit.*, p. 388.

‡ *Loc. cit.*, p. 375.

ing Government bureaus and commercial companies, are using film with a great deal of success. As far as the writer knows all the more recent designs of aerial cameras use films. On the other hand, all optical-mechanical mapping processes are committed to the use of plates because of the small emulsion displacements mentioned by the author, to which film is subject at times. The errors in the position of photographic images resulting from this source, as well as those due to lens distortion, are only of academic interest in the preparation of maps by non-mechanical methods, because these errors do not average more than the width of a line, which, after all, is the limit of accuracy in drawing a map.

The author speaks disparagingly of the mosaic type of aerial map, stating that it has been called "a caricature of the landscape", and that "the weakness of a mosaic, fatal to its use as a substitute for the engineering map, is its lack of uniform scale".* The writer is fully aware of the shortcomings of the mosaic, but believes in looking the facts in the face. Briefly stated, the demand for mosaics on the part of engineers is far greater than for line maps. Engineers are not averse to using maps known to contain errors, having learned through experience to place but little trust in the scaleability of any kind of map. The discrepancies inherent in mosaics have proved, as a rule, trivial alongside the blunders and omissions inherent in maps made by ground surveys. Mosaics afford information for engineering as well as commercial purposes that the best line map cannot supply, and convey, as a rule, infinitely more to the engineer's client. This latter consideration is paramount.

The author's remarks apply more particularly to topographic maps for reservoir sites and for other purely engineering purposes, which must necessarily be of the line-map type with contours. These form but a small fraction of the demand for aerial maps. Speaking from intimate contact with the aerial mapping industry, the writer has learned that the bulk of aerial maps are for land development, transmission line location, tax assessment, city planning, zoning, oil exploitation, lawsuits, timber surveys, and many minor purposes. Such maps are nearly always ordered through, or at the recommendations of, engineers and these orders, in most cases, specify mosaics.

The author states† that "the ideal method * * * will make maps directly from the camera records, and dispense with ground measurements or reduce these to a minimum needed for control only." If this be true, then the process described by him still is far from ideal, for its characteristic in common with that of other optical-mechanical processes is that it requires a great deal of ground control. It does obviate ground surveying for locating contour lines, but contour lines do not appear to be essential in the production of maps to the extent that they did before the advent of aerial surveys.

The greatest demand for map-making exists at present in South and Central America, where vast areas are so densely covered with vegetation as to make ground surveying for the purpose of establishing control points, such as described in this paper, a practical impossibility. For the same reason, contour location by parallax methods becomes a doubtful expedient, because over

* *Proceedings*, Am. Soc. C. E., March, 1926, Papers and Discussions, p. 383.

† *Loc. cit.*, p. 368.

large areas the ground cannot be discerned in the photographs. It would be interesting to learn from the author what success has been had with this method in such regions.

From a general survey of the use of optical-mechanical methods it would appear that they are best adapted to the production of large-scale contour maps in regions where an abundance of ground control can be obtained cheaply. This is the case in European countries, where cadastral maps of great accuracy are available for use as ground control. This feature has had much to do with the success attained by optical-mechanical mapping processes in those countries. To what extent such methods will be found economical in the Americas, where ground control as a rule is conspicuous by its absence or difficult to resurrect, remains to be seen.

LAND SETTLEMENT OF IRRIGATION PROJECTS

Discussion*

By Augustus Griffin, M. Am. Soc. C. E.†

Augustus Griffin,‡ M. Am. Soc. C. E. (by letter).§—The writer is pleased with the frank discussion of this paper and wishes to express his thanks to those who have contributed to it. It is not a simple subject either to outline or to discuss. The details of land settlement and its attendant problems vary widely on different projects.

The one fundamental fact brought out is that the problems are comparatively simple if the land is worth the price. Quality of soil, price of land, annual costs of operation and maintenance, and costs of land preparation are relative and are of greatest significance in relation to products and markets. Time is an important factor; the poor project of to-day may be the good project to-morrow.

One of the highest costs paid by the pioneer settler may be isolation—the lack of social intercourse and the lack of educational and cultural opportunities for a growing family. The most pathetic regret the writer has heard was expressed by a man who had lost the substantial savings of his active life in an agricultural venture in a pioneer section and was reduced to day labor as a means of livelihood—a regret that his children had passed through the formative period without schooling and without the cultural advantages of a settled community. The same regret has been expressed by others who achieved a measure of financial success under similar conditions. For the same reasons others have given up before financial considerations became a factor. Undoubtedly, any settlement should be "close" settlement in small units.

The prevailing opinion of the day is that ample land is now available to supply present demands and that new projects should be undertaken with great caution. It must be recognized, however, that there are strong inducements for owners of large tracts of land to undertake colonization. The pressure of accumulating interest and increasing taxes may be very great. There is genuine need for National development whether or not it be actually as pressing as it may seem. There are good reasons why the pioneer settler may consider himself the benefactor.

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* Discussion on the paper by Augustus Griffin, M. Am. Soc. C. E., continued from September, 1926, *Proceedings*.

† Author's closure.

‡ Supt. of Operation and Maintenance, Eastern Section, Irrig. Block, Dept. of National Resources, C. P. Ry., Brooks, Alberta, Canada.

§ Received by the Secretary, October 15, 1926.

CORROSION OF CONCRETE

Discussion*

By Walter D. Binger, Assoc. M. Am. Soc. C. E.

Walter D. Binger,† Assoc. M. Am. Soc. C. E. (by letter).‡—In the winter of 1919 and spring of 1920 a river structure was built in the Housatonic River at Shelton, Conn. This structure consisted of a beam-and-girder deck, supported by bents comprising four piles. Reinforced concrete was the only material used except for the piles, which were of wood. During low tide several feet of the wood piles were exposed, while during high tide the concrete columns were submerged for several feet. The Housatonic at this point has a velocity of approximately 3 miles per hour.

Between April and August, 1922, a second smaller section, 366 ft. long, was built, similar in all respects to the previous structure, and of identical materials. Before starting work on the second section the first was examined and found to be in good condition. The materials consisted of new billet structural grade reinforcing bars, standard Portland cement, a pre-mixed sand and gravel dredged from Long Island Sound near Port Jefferson, N. Y., and city water. The cement and steel were tested by a public testing laboratory and found to conform to the specifications of the American Society for Testing Materials.

What have been called the first and second sections were built by the same construction company with the same general superintendent, but under different resident superintendents. The river structure in question runs along property occupied by a textile manufacturing plant, which included a large dye-house emptying its waste into the river. Between February and April, 1923, this dye-house was wrecked and a new one was built in an adjoining location also at the "river driveway".

On April 15, 1923, the writer was notified that both the first and second sections had deteriorated. Inspection showed that the concrete had become so soft that large pebbles could be picked out by hand with little effort. It appeared to be badly abraded at the corners where the reinforcement had become exposed.

It was desired to determine as nearly as possible the true cause of this deterioration, and much money was spent for tests, examinations, and expert opinion. A chemical analysis of the river water in the vicinity of the structure was obtained, based on about one hundred samples taken at different times.

* Discussion on the paper by John R. Baylis, Assoc. M. Am. Soc. C. E., continued from November, 1926, *Proceedings*.

† Pres. and Treas., Thompson & Binger, Inc., New York, N. Y.

‡ Received by the Secretary, October 20, 1926.

A chemical analysis was made of the deteriorated concrete, with results as herein given. However, no analyses could be made of the water as it was between February 2 and April 15, which is the period during which the concrete in both sections failed.

Apparently, the gray Portland cement had been converted into a white powdery substance with no binding qualities. The appearance of the reinforcing steel at the time the corrosion of the concrete was discovered, was such as to preclude the possibility that the damage was caused by corrosion of the steel. Furthermore, in each case the deterioration of the concrete was from the outside in and never in the form of smooth plates of hard concrete spalled off from the inside in the typical manner of a concrete failure due to steel corrosion.

The concrete was mixed in the proportions of 1:1.5:3 in a small standard mechanical mixer. The water was measured in pails. No effort was made to grade the aggregate for a particularly dense mix, but the ready mixed aggregate was well graded from fine to coarse in accurate proportions of one of sand to two of pebbles.

The steel was placed 1½ in. from the surface with a fair accuracy which in a few cases brought it quite close to the surface, and in others as far as 3 in. The forms were made of tongue-and-grooved lumber, well oiled, and tight enough to hold mortar. The pile-cap forms were of 2-in. planks, open at the top and giving access to the river water as soon as the tide covered the forms several hours later.

The deterioration (Fig. 17) was at its worst in some places where the steel was even 3 in. from the surface. At the date of last examination, in the fall of 1925, the superstructure, namely, that part above high water, was in essentially perfect condition (Fig. 18).

While deterioration of the concrete was rapid during the period of several months after its first discovery, it subsequently became less noticeable, and during the early part of 1925 seemed to have ceased. The deteriorated concrete was easily affected by frost and floating objects. There seems to be no possibility that frost and the abrasion by floating objects were the primary causes of the deterioration, as the first section showed no ill effects of the cold winters of 1920 and 1921, while it corroded in the same period of two months during which the second section deteriorated.

The structure was provided with several expansion joints extending from top to bottom. In at least two cases this was not sufficient to take up the total expansion and some of the concrete at the edges of the abutting bents was cracked. Pieces so broken off showed sound concrete, in no way resembling the soft, almost mushy, part of the structure deteriorated in the water. The most minute examination of the entire structure and the study of several hundred photographs failed to disclose a single instance of a crack due to strain, overloading, or lack of steel; in fact, the only cracks in the entire structure were at a few "day's work" joints where the beams of one bent had been continued into the adjacent bent half-way between columns. For architectural reasons, the columns and most of the members were made much

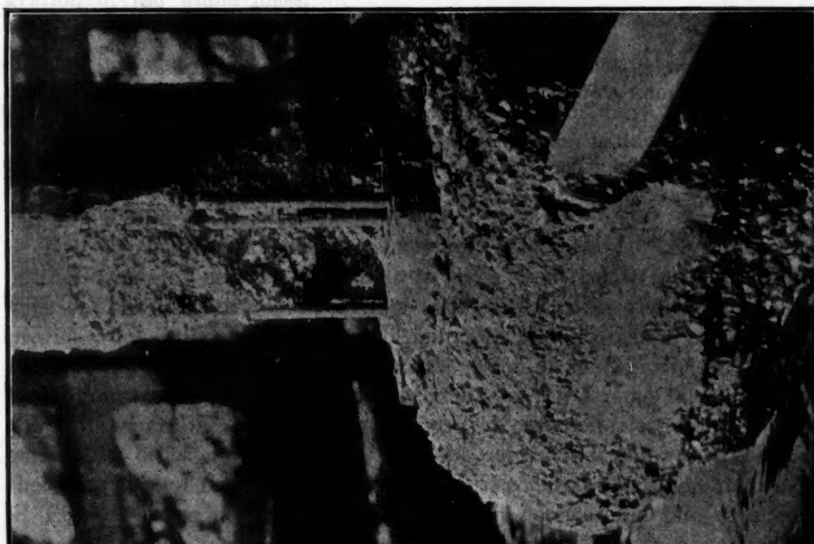

FIG. 17.—VIEW OF STRUCTURE SHOWING DETERIORATION OF PILE CAPS AND COLUMNS.

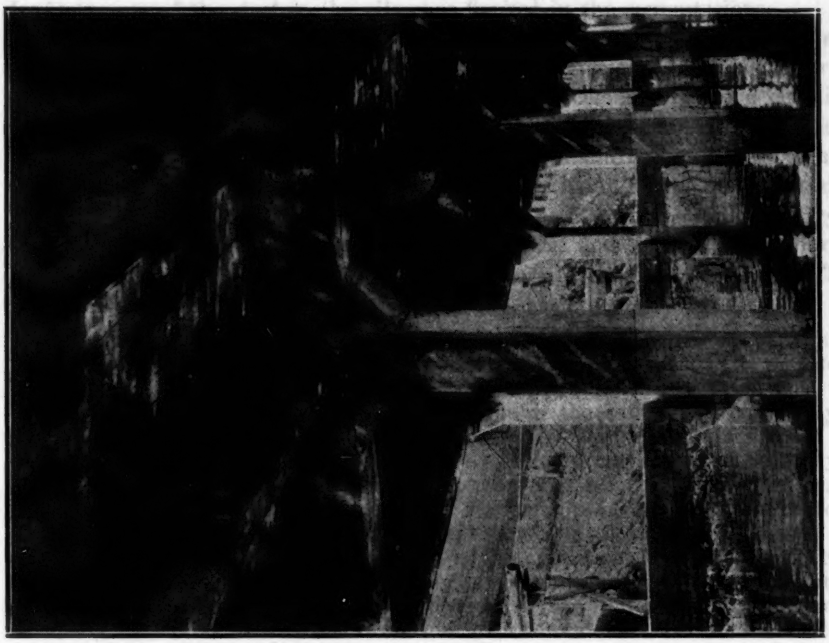

FIG. 18.—VIEW OF STRUCTURE ABOVE MEAN HIGH WATER, SHOWING GOOD CONDITION.

A chemical analysis was made of the deteriorated concrete, with results as

the forms several weeks later.

The deterioration (Fig. 17) was at its worst at some places where the steel was over 3 in. from the surface. At the time of last examination, in the fall

been continued into the adjacent beat half-way between columns. For architectural reasons, the columns and most of the members were made much

larger than the light live load required, and the steel had very low stresses. In fact, none of the steel in the pile caps figured in the computations.

It is the writer's opinion, based on his own observations and a long and careful study of the facts, that the cause of the deterioration is the destruction of the cementitious qualities of the Portland cement, due to a chemical reaction with the substances carried into the water for the two months in which the dye-house was rebuilt, during which time the manufacturer failed to neutralize his acids. This belief was arrived at after having access to the studies made by the following experts, all of whom concur in this opinion: Bryan C. Collier, Benjamin A. Howes, E. DeV. Tompkins and Watson Vredenburgh, Members, Am. Soc. C. E., and Louis Weisberg, Consulting Chemist. The report of Dr. Weisberg includes the following statements:

"A typical sample of the deteriorated concrete taken from below high water in June, 1924 (the year after the deterioration was first reported) showed a white furry deposit, which could be crumbled easily between the fingers and which was found to extend through the sample. Upon analysis this white material dissolved in hydro-chloric acid with production of carbon dioxide and was found to be mainly calcium carbonate. The analysis of this concrete sample shows that the cement has been replaced by a material in which calcium carbonate is one of the principal constituents. Approximately, 80% of the calcium has been transformed into calcium carbonate.

"Analyses of the water samples show that practically every sample contains bicarbonates. A certain number also contained carbonates. Either carbonates, bicarbonates, or carbon dioxide in the water would be capable of bringing about this transformation of the calcium in the cement to calcium carbonate. The water samples for the most part also show the presence of acid which is capable of dissolving the calcium carbonate formed within the mass and removing it therefrom, thus bringing about disintegration of the whole mass."

Regarding the water, he says:

"Of the samples taken Saturday and Sunday, 33% showed an alkaline reaction, whereas those taken on working days only 10½% showed alkaline reaction, and 89½% showed an acid reaction.

"In short, it is my opinion that the disintegration is due to the transformation of the calcium in the cement, to calcium carbonate by the action of carbonates and bicarbonates in the water, followed by a solution of this calcium carbonate by the action of acid in the water."

Dr. Weisberg also made a comparison of a complete chemical analysis of the sample of deteriorated concrete taken from below the high-water line with one of undeteriorated concrete taken from above that line. The piece previously mentioned as being broken off at the expansion point, served as the latter. He found only 24% of the calcium in this air-set concrete to have been transformed to carbonate, while in the sample taken from below the high-water line, calcium in the form of carbonate is nearly 80% of the whole. In other words, not only can the acid attack the vulnerable part of the concrete, but three times as much of the concrete is vulnerable.

Fig. 17 shows in detail one of the worst deteriorated pile caps and columns, the front face of the column being corroded up to a horizontal line which coincides very closely with mean high water as evidenced by a horizontal deterioration on the adjacent face. At this time the structure was

carrying a load of broken stone equalling approximately twice the designed load per square foot. Although this load remained for a long period, the structure showed not the slightest sign of suffering from this severe test.

A close view of the structure above mean high water is given in Fig. 18 which plainly shows excellent concrete. This photograph was taken at mean high tide.

STREAM REGULATION WITH REFERENCE TO IRRIGATION AND POWER

Discussion*

By J. C. Stevens, M. Am. Soc. C. E.†

J. C. Stevens,‡ M. Am. Soc. C. E. (by letter).§—The writer is gratified at the discussion his paper has brought forth.

Mr. Sargent‖ has presented very valuable data on the practical regulation of reservoirs for flood control and water power without forecasts of probable supplies. Doubtless, the same principles can be utilized in many cases in practical regulation for power and irrigation. The storage capacity above spillway level probably can be counted upon and the spillway capacity reduced accordingly in large reservoirs for which complete records of run-off are available. On small reservoirs, however, where stream-flow records are rather "sketchy", it would be safer not to count on it.

The writer wishes every engineer who has to do with the development of natural resources could read Mr. Hoyt's discussion.¶ It is a clear, concise statement of the policy that must govern the practical administration of public land and water resources. As Mr. Woolley** points out, conditions may arise that make power development paramount to irrigation. Every water-shed must be studied in the light of the most economic use of its water supply. Behind the conclusions drawn from such an analysis there must exist unprejudiced authority to enforce them in the interest of all concerned, and, obviously, this function is best exercised by the Federal Government.

Mr. Galloway†† questions the writer's diagrams because they express water supply and demand in percentages of totals. To have given quantitative data would have defeated the purpose of the paper, which, as stated in the Synopsis‡‡ and elsewhere, was to inquire into the subject in general terms. Quantitative figures would have made a local problem of each example cited, and the study would have been limited to the present demands. By treating these examples generally, a forecast is gained as to the future ultimate utilization of the entire annual supply.

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* Discussion on the paper by J. C. Stevens, M. Am. Soc. C. E., continued from November, 1926, *Proceedings*.

† Author's closure.

‡ Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

§ Received by the Secretary, October 18, 1926.

‖ *Proceedings*, Am. Soc. C. E., May, 1926, Papers and Discussions, p. 1036.

¶ *Loc. cit.*, p. 1040.

** *Loc. cit.*, p. 1046.

†† *Loc. cit.*, November, 1926, Papers and Discussions, p. 1833.

‡‡ *Loc. cit.*, April, 1926, Papers and Discussions, p. 614.

Fifty or a hundred years hence these streams will be flowing much as they are to-day. In all probability the curves of demand for power and irrigation will not be materially different from those indicated, because they are expressed in percentages of the total rather than in second-feet or kilowatts. The simplicity and value of presentation in the manner adopted by the writer would have been completely lost had he followed Mr. Galloway's suggestion as illustrated by Fig. 14,* in which neither percentages nor quantities are given, the curves being merely "typical".

The writer reiterates his statement that water is and always will be vitally necessary for irrigation, but not in the same degree for power. There is stern justice in conceding a preferential use to agriculture, and power interests will benefit in the long run by such a concession. Whether or not there is a conflict of interest is a purely local fact. There is nothing hypothetical about it. The very fact that power companies in California and elsewhere are constructing storage reservoirs and power plants above irrigation reservoirs is proof that a conflict of interest did exist, and that the solution lay in a double system of reservoirs. Many sections, however, are not so topographically fortunate in this respect, and where such solutions are not possible concessions will have to be made on both sides. There is, however, a limit to the concession irrigation can make.

Those discussors who mentioned the subject at all agree with the writer in his remarks about storage by forestation. It is gratifying to know that some engineers at least have done some original thinking on this subject and have penetrated the thick coat of propaganda spread over it.

* *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1834.

INCREASING THE EFFICIENCY OF PASSENGER TRANSPORTATION IN CITY STREETS

Discussion*

By Messrs. J. P. Snow, Theodore T. McCrosky, and F. Lavis.

J. P. Snow,† M. Am. Soc. C. E. (by letter).‡—The introduction of motor vehicles on highways is giving transportation a "jolt" comparable in some degree to that given it one hundred years ago when George Stevenson drew a train of cars on iron rails by a steam locomotive. The jolt affects people in many ways, and severe street congestion in cities is one of them. The author suggests the elimination of parking and goes so far as to recommend keeping private cars off certain streets, with the idea of confining the use of these streets to busses and electric cars.

Public highways were designed and built for free use by private vehicles, and owners of these vehicles have a natural right to drive them into town if it is profitable for them to do so; they, and not the public, are the judges as to this profitableness. The right of any person to park a vehicle on the highways so as to interfere with their legitimate use by others is another matter, wholly subject to public regulation. It is probably useless to consider prohibiting private cars from being driven over streets on which busses and other carriers are allowed. Without doubt efficiency in the number of passengers accommodated in a given time could be obtained by busses; but efficiency of this sort is not the only factor to be considered.

The elimination of parking will not prevent congestion because this occurs at intersections where there is no parking. The chief objection to parking anyway is that it prevents the objector from committing the same offense himself. Other means of handling the problem than by these two suggestions are certainly pertinent to the discussion.

The worst congestion occurs at intersections, due to the crossing of streams of traffic at grade. Separating grades is awkward, unsightly, and expensive. The present stop-and-start method causes long delays when long queues of vehicles have to be accommodated. In fact, the volume of traffic is reduced by this method to less than one-half the possible clear-street capacity if both streets are equally used.

What may be called the zigzag system will permit constant movement by vehicles on all streets except an occasional short stop to allow pedestrians to cross. A large flock of pedestrians can be accommodated in a fraction of the time required when strings of vehicles must pass, as now.

* Discussion on the paper by John A. Miller, Jr., Assoc. M. Am. Soc. C. E., continued from November, 1926, *Proceedings*.

† Cons. Engr., Boston, Mass.

‡ Received by the Secretary, October 9, 1926.

The first requisite under the zigzag system is that all streets must be rigidly restricted to one-way traffic; the second is that streets in one direction be selected as principal thoroughfares or arteries and those crossing them be considered as cross-streets. Movement on the arteries shall be in one direction throughout their length, while that on cross-streets shall reverse at each crossing of an artery. Fig. 10 shows a checker-board layout comparable to several of New York's avenues and streets.

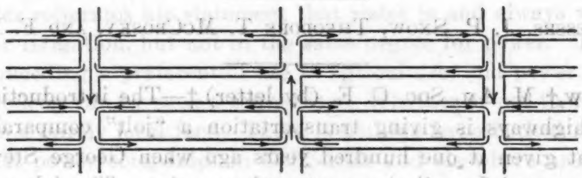

FIG. 10.

This system allows no direct crossing of the path of one car by another car. Merging into a line of moving cars is far different from crossing that line. The cross-town car merges into an artery lane, slides over into the next lane in traversing the block, turns into the next cross-street, and at the next artery turns back to enter again the street that it just left. If the blocks are square the distance traveled by the cross-town car is twice the direct distance actually accomplished; but if stops for pedestrians are avoided, the time will be less than is required by the stop-start system. In the New York street layout (Fig. 10), the distance traveled by the cross-town car will be 1¼ times the air-line distance.

Pedestrian crossings under this system will be more efficient than at present because all vehicles will be stopped and the whole intersection will be clear, so that the four crosswalks and the two diagonals can be used simultaneously. Under this system and with one-way traffic under any system, only one-half the street corners need to be rounded. The rounding should be generous, with radii of at least 12 or 15 ft. which should be insisted upon whenever a corner is rebuilt.

In truly congested sections of a city where the zigzag system of cross-town traffic is required no surface rail traffic can be tolerated. In a congested section rails must be either overhead or in subways. Surface rails in streets are suitable only in suburbs.

With continuous movement an enormous traffic can be accommodated. Where pedestrian stops cannot be tolerated, overhead crossings with escalators can be installed at a cost that would not be prohibitive. Speed and other details of regulations should be established and enforced. A single lane of moving vehicles will be ample for some streets, a double lane will be needed in others; in but few streets will all the roadway between curbs be needed for legitimate movement. At reasonable speed it should always be possible for a car to stop for one or two people to enter or leave; longer stops should be prohibited. Lanes for traffic should be plainly defined by paint lines on the pavement. In many streets these lanes can be arranged to leave parking space

either at one or both sides against the curbs and in exceptional cases in the center.

A distinction in terminology should be made between "ranking" cars parallel to the curb and "parking" them at an angle. From the curb to the traffic boundary 8 ft. is sufficient for ranking and 18 ft. for parking. The latter is preferable, and if on only one side of the street it should be the right-hand side as traffic flows, with regular stall lines painted on the pavement. These stalls may be at angles of 30 to 45° with the curb, and should be entered head on from the traffic lane whichever side of the street they may occupy.

If ranking or parking space is provided there is no logical reason why the city should not derive a revenue from it. The highways are provided for moving vehicles and not for storage purposes; but when storage can be allowed without sensibly delaying traffic it is wise to allow it under regulation and at a price. No car owner can claim an inherent right to occupy street area for free storage of his car; hence he should pay for the privilege or stay away. Police are now employed to tag cars that stop too long at a given place, and they could as well collect the prescribed toll. Bus lines could rent standing room permanently for their regular stopping berths at which to discharge or receive passengers and abutting merchants could rent space for the shipping and receipt of goods.

The one-way feature here recommended, the adoption of which the writer believes to be imperative in order to get the greatest efficiency in street service, reduces the necessary width of streets materially. Sidewalks should be widened where needed. When streets are repaved the space outside the traffic lanes may well be surfaced with cheaper pavement than is required by traffic. Good bituminous macadam is ample for these areas.

Where real rapid transit systems are in operation it would seem rational for a driver to park his car in some field arranged for storing near a suburban station of the system and go into town on a train. The fares plus the charge for outdoor storage could hardly be more than a fair charge by the city for street parking, and the time and risk should be materially lessened. Busses or other carriers operating on the surface cannot properly be called rapid transit.

THEODORE T. MCCROSKY,* JUN. AM. SOC. C. E. (by letter).†—This paper is an excellent presentation of the present traffic situation in the larger cities, and suggests valuable methods of aiding in the solution of the problem of congestion.

The writer believes that the discharge capacity of city streets is determined not so much by their width as by the size and complexity of their intersections. Street intersections must be enlarged. In busy sections, a great enlargement in most cases would be prohibitive in cost, even were it physically possible. Fortunately, there is a means of increasing the size of an intersection which comes under the head of minor changes.

It was found by experiment that the turning radius of a certain automobile having a 132-in. wheel-base was 17 ft. 4 in. Many cars have an even

* Instr., Sheffield Scientific School, Yale Univ., New Haven, Conn.

† Received by the Secretary, October 19, 1926.

longer radius. Furthermore, when a car turns a corner, it does not follow a circular curve, since the steering wheel must be turned gradually. In order to avoid the necessity for the car to swing out toward the center of the street it is entering, and thus waste roadway space in the intersection, not only should the curb corners be of long radius, but also, when feasible, of varying curvature. Such curves would be difficult to locate, and a simple but satisfactory substitute plan is illustrated in Fig. 11. The 25-ft. radius has been chosen for purposes of illustration, as representing the ideal when sidewalk widths and intensities of pedestrian traffic permit its use.

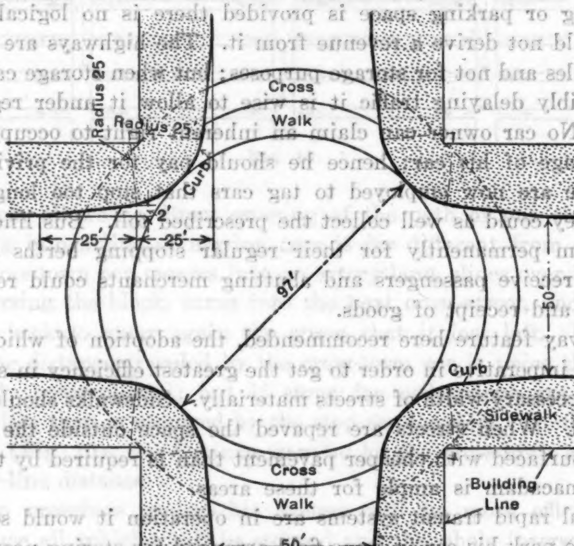

FIG. 11.—STREET INTERSECTION. PROPOSED METHOD OF LOCATING CURB CORNERS AND CROSS WALKS.

In the case of heavy pedestrian traffic, this type of curve would cut off too much sidewalk space, but in no case should the curb corner be cut back on a radius less than the width of the narrower sidewalk; and a radius equal to the width of the wider sidewalk is usually practicable. When pedestrian traffic is fairly light, the method illustrated can always be used; and it can also be employed when it is possible to pass pedestrian traffic under the corner of the building. This is done by cutting back the corner of the building on the first floor, and supporting the upper stories on a column. A great many shops now provide this facility in order to gain the advantage of having their door directly on the corner.

With curb corners cut back as illustrated in Fig. 11, cars making right-hand turns would be able to hug the curb all the way. It would be advisable to prohibit the ranking and stopping of vehicles for an appreciable distance back from the curb line of the cross street. This restriction gives an extra traffic lane on each side of each street at the intersection, which is precisely where extra space is most needed. At the intersection of two 50-ft. road-

ways, the diagonal distance between curb corners cut on 6-ft. radii is 75.6 ft. When the radii are increased to 25 ft., this diagonal distance becomes 91.3 ft. This, again, shows how the size of an intersection may be increased by this comparatively simple means. By the curve and tangent method shown in Fig. 11, the diagonal distance is increased to approximately 97 ft.

It has been argued that long radius curves make it harder and more dangerous for pedestrians to cross the street. The writer feels that this objection does not hold when cross-walks are marked out in curves (Fig. 11). Furthermore, observation at the corners in the vicinity of Times Square, New York City, indicates that small radii are more pernicious than large. The pedestrian traffic in this district is colossal in the evenings on account of the theater crowds. Instead of waiting on the sidewalks, where they should, pedestrians stand in the roadway, about 3 ft. from the curb, knowing from experience that most automobiles have to swing the corner at least that far from the curb. Then an automobile approaches which can turn on a 12-ft. radius. The pedestrians try to back up and get back on the sidewalk, but are unable to do so on account of the crowd behind them. Many accidents result. If the corners were cut back so that the majority of automobiles could follow them closely, in a short time the pedestrians would realize that vehicles practically always turn close to the curb, and would soon learn better than to stand in the gutter as they do now.

Cross-walks should be curved as illustrated in Fig. 11. All arcs are circular. In this way, the pedestrians are kept a little farther from the congestion in the intersection, are less in the way, and run less danger of being hit by cars turning the corners. At intersections at which is installed the rotary system of traffic regulation, it is particularly desirable to lay out the cross-walks in this form.

The writer is of the opinion that it would not be fair to prohibit all ranking along street curbs, nor would it be possible. A driver would still retain the privilege of stopping for the instantaneous picking up or depositing of passengers, or for the expeditious handling of goods; and such stopping, as has been pointed out by numerous authorities, would rob the street of a lane of moving vehicles for some distance in front of and behind the standing car, as operators will not drive next to the curb for fear of getting "boxed" behind a stationary car. It is none the less perfectly fair to refuse ranking space to the selfish driver who wishes to leave his car on the public street all day long. If a pedestrian likewise elected to stand on the sidewalk at a busy corner, a policeman would tell him to move on. All drivers must have the privilege of stopping at the curb for a reasonable time, except close to street intersections, and this privilege becomes impossible of realization when certain drivers elect to block the curb all day with "dead" vehicles.

The relief of traffic congestion is an engineering problem, and the solutions of it must be economic solutions. Such ultimate means of relieving congestion as sidewalks arcaded on the second story, and, eventually, buildings standing on stilts, thus providing running and parking space beneath them for vehicles, are bound to come. Engineers and architects would do

well seriously to consider providing some such facilities in future buildings. The writer agrees fully with Mr. Miller that, in the meantime, traffic engineers can do much to increase the efficiency of transportation in city streets without materially altering the existent curb and building lines.

F. LAVIS,* M. AM. SOC. C. E. (by letter).†—The writer is much more nearly in accord with Messrs. Stevens, Lewis, Slattery, and de Blois,‡ discussors of the paper, than with the author, in so far as they point out the fact that the capacity of many streets or highways is limited to a far greater degree by obstructions such as cross traffic, etc., than by parking.

There may be some exception to this (in degree) in the case of certain "Main" streets in country towns or villages, where main-line highways pass through and where the cross traffic may be light and parking the real obstruction, but in city streets, especially in New York—around which the discussion seems to have centered—the cross-traffic interruptions are much the more serious factors in decreasing the efficiency and capacity.

In connection with the study of certain economic factors governing the design of the location of the main-line, through highway that the State of New Jersey is building through the congested Metropolitan Area on the west side of the Hudson River, calculations were made by the writer and S. Johannesson, M. Am. Soc. C. E., of the costs of delays at a street intersection of two main-line highways, each capable of carrying four lanes of traffic.

It was assumed that the main through highway would have a traffic of 20 000 000 vehicles per year and that at the proposed crossing the main-line traffic could proceed for 3 min. and the cross traffic for 1 min., alternately. It was further assumed that there would be measurable loss during only 15 hours each day, when traffic was regulated, and that the loss on two lanes in one direction would be only about one-third that of the other two lanes on the highway moving in the direction of maximum flow of traffic.

Without going into details of the computations, it may be stated that the actual loss in car-minutes per day was calculated to be:

On the main highway....................	14 532 car-minutes
On the cross street....................	9 000 " "
Total	23 532 car-minutes

Assuming an actual loss of 2.2 cents per car-minute (an average for the proportion of various types of vehicles assumed to use the main highway), the total monetary loss was calculatel to be $156 000 per year, equal to a capitalized value of about $3 000 000.

Inasmuch also as in this particular case the State of New Jersey is building, at a very considerable cost, this new highway to take care of the traffic increase in this territory, the question of traffic capacity is an element of importance, and it was found that the delay at this assumed crossing had a monetary value of another $3 000 000 due to the decrease in capacity on account of the delays. These calculations are believed to be conservative, and can be

* Engr., State Highways, Jersey City, N. J.

† Received by the Secretary, October 29, 1926.

‡ *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1444 *et seq.*

amply justified. They tend to confirm the statement made at the beginning of this discussion in regard to the importance and cost of the delays due to cross traffic.

The only solution of the traffic congestion problem which appeals to the writer is that now being put into effect in New Jersey on the highway referred to, namely, the entire separation of through from local traffic. It involves large expenditures of money, but the most conservative estimates of losses to the operators of motor vehicles will show in many, if not all, of these cases that there is hardly any expenditure for this purpose which may be considered at all within reason, but what can be justified from an economic point of view.

SNOW REMOVAL PROBLEMS OF ORGANIZATION AND OPERATION

Discussion*

By W. A. Van Duzer, M. Am. Soc. C. E.

W. A. Van Duzer,† M. Am. Soc. C. E. (by letter).‡—Snow removal, to a greater extent than any other highway activity, is affected by local conditions. In fact, generalization is difficult in dealing with this topic, since the work in one locality or State is likely to be so different from that in another.

The scope and importance of snow removal are determined chiefly by the amount of snowfall, which measures the initial level depth to be disposed of; the extent of drifting, which may increase the depth by 40% and which aggravates the removal problems and necessitates the handling of quantities in excess of the level-depth snowfall; and the volume of traffic, which to a considerable extent determines the necessity of, and the justifiable expenditures for, snow removal.

An idea of the amount of snowfall can be obtained from annual averages as reported by the United States Weather Bureau. In considering averages, however, it is necessary to bear in mind that they change from year to year, and in any one year a State average may represent wide variations. For instance, the annual snowfall for the State of Pennsylvania for the winter of 1923-24 averaged 48.0 in.; for the winter of 1924-25, 43.1 in.; and for the winter of 1925-26, 50.4 in. The 1925-26 average was computed from forty-nine stations ranging from a minimum of 17 in. in the southeast corner of the State to a maximum of 137 in. in the northwest corner.

The extent of drifting is more difficult to visualize. There are so many variable factors that for future reference the safest way to study causes of drifting is to go out on the road itself after the drifts have formed. Even then, it must be considered that in the next storm the wind may come from a different direction and create different drifts. A fair realization of the possibilities of drifting, however, can be obtained from a study of a chart showing the topography, direction, and force of prevailing winter winds, and the monthly mean temperatures. The topography will indicate the extent of grading, in which cuts and side-hill sections are significant; the direction and force of prevailing winds will show which roads will catch most of the cross-drifting and how serious the drifting is likely to be; and the temperature data will indicate, to some degree, how long the snow may be expected

* Discussion on the paper by George E. Hamlin, M. Am. Soc. C. E., continued from September, 1926, *Proceedings*.

† Deputy Eng. Executive, State Dept. of Highways, Harrisburg, Pa.

‡ Received by the Secretary, October 2, 1926.

to lie along the road, without crusting and dry enough to continue subject to drifting. Greater care is now being exercised in the location of highways to minimize the danger of drifts.

The volume of traffic naturally varies with the density of population adjacent to the road, and, therefore, in the absence of traffic-flow data, if the snow-removal conditions of one section are known the indices of density of population will furnish a good basis of comparison for any other section.

Although fundamental conditions present such wide variations, there are two principal problems which concern all States in which any considerable snow removal is to be undertaken. The first of these is to keep the road open continuously. It would be comparatively easy to remove snow if there were no time element. It is the necessity of removing the snow before it has accumulated to such an extent as to block traffic that makes snow removal a serious and costly proposition. In Pennsylvania, all roads are considered open for travel by the State office; and information is given to the public unless advised by the field that the road is closed, in which case an estimate of the time the road will be closed is included in the report.

The second is the always present problem of costs. It is necessary to hold the cost of the work down to a minimum. At the same time, it is necessary to make the accounting complete so that other lines of activities are not inequitably burdened with charges. The State of Pennsylvania during the past years has been practically subsidizing snow removal by charging truck and tractor equipment at the same rental rates as on other road work. It has become necessary to increase these rates by an average of 60% for snow-removal work on account of the heavy repairs and depreciation that attend the use of tractors and trucks in plowing snow.

Neither of these two problems can be considered separately; both resolve themselves into one problem of efficiency. Viewing snow removal in this way, several points present themselves, such as, program, organization, location, snow fence, equipment, personnel, operations, and costs.

Program.—The first step in snow removal is laying out the program. This must be done with due regard both to the budget and to traffic requirements.

The most readily apparent test of the economics of snow removal is from the standpoint of the protection of investment. The fixed charges, depreciation and interest, on a high type of surfaced road, for instance, will be about \$4 000 per mile per year, or more than \$10 per mile per day. If winter and summer density of traffic were assumed to be equal, it would be economical to spend for snow removal as much as \$10 per mile for each day that the road would otherwise be blocked. Or, assuming the winter density of traffic to be normally one-half of the annual average, the justifiable expenditure, from the same standpoint of protecting investment, would be \$5 per mile for each day that snow would block the road. Under the second assumption, in a locality where snow would block the road 60 days a year, an expenditure of less than \$300 for snow removal, if it kept the roads continuously open, would evidently be advantageous.

The same line of reasoning applied to lower types of hard surfaced roads, representing smaller first cost, would, of course, produce lower figures, perhaps as low as $1.50 per day; but in the case of these roads, there is an additional factor to be taken into account. Winter snow removal undoubtedly lowers the cost of spring repairs. Therefore, it would be fair to group all roads that carry an annual daily average of more than 500 vehicles and assume for them the same value of snow removal as for the high type surfaces.

Whether or not the investments and saving in repair costs are considered, the final check on the snow-removal program is to see if it is within the budget, if it is practicable from the standpoint of forces and equipment available, and if the mileage is laid out to best serve the localities and the traffic. Snow removal in Pennsylvania costs about 40 cents per motor vehicle registered.

Organization.—The organization for snow removal should be designed to cope with the particular conditions it will have to handle. Mr. Hamlin's description* of such an organization indicates a highly centralized control. Under the conditions with which the writer is familiar, the work is such as to require decentralization of control for efficiency.

The Pennsylvania program comprised more than 5 000 miles last winter (1925-26), and will be about 6 000 miles for the coming winter (1926-27). Some of the work is more than 300 miles distant from headquarters, so that the plan of organization to insure unimpeded operation is:

1.—Central headquarters at Harrisburg.

2.—The State divided into four Engineering Divisions, approximately the northwest, northeast, southwest, and southeast quarters of the State; a Division Engineer resident in each, reporting to Harrisburg.

3.—Each Engineering Division divided into three or four Engineering Districts (fifteen Engineering Districts in the State); each Engineering District in charge of a Resident District Engineer reporting to his Division Engineer.

4.—Each Engineering District divided into three or four Maintenance Districts (fifty-two Maintenance Districts in the State); each Maintenance District in charge of a Superintendent reporting to his District Engineer. The Superintendent is in direct charge of operations. The snow-removal program mileage is divided into sections, each designated by a letter, beginning with "A" in each Maintenance District, and equipment assigned to care for each section.

A snow chart, kept at Harrisburg (like the chart described by Mr. Hamlin† posted from wired reports from the superintendents to show any road that may be temporarily blocked or open only half width) is used for disseminating information to newspapers and for administrative control, but in each of the fifty-two districts similar control is kept by the superintendent, who remains at his post of control during the critical period of snow-removal operation. The maintenance superintendents have assistants who follow up the equipment and crews on the roads seeing to the actual details of the work and taking care of repairs or any emergencies that may arise. They and

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1121.
† *Loc. cit.*, p. 1122.

the equipment operators report to the superintendent by telephone from time to time as they reach certain objectives or require advice or assistance.

The Pennsylvania system for the distribution of equipment differs somewhat from the one described by Mr. Hamlin* in that the tractors and trucks are not assigned to various foremen who are in charge of the maintenance of certain sections of road, but operate from one or more centrally located points (State garages or storage sheds) in each maintenance district; and the equipment operators report directly to the superintendents. The trucks and the tractors are charged with the responsibility of maintaining two-lane traffic ways. The foremen and caretakers in charge of the various sections are not all on regular snow duty, although some of them serve as equipment operators or helpers, but they are subject to call for emergency work and after each storm they are required to clean up the roadway; and open culverts, and side and shoulder drains.

The superintendents' control of snow removal in Pennsylvania is facilitated and assisted at need by co-ordination of equipment and supplies under the direction of the district engineers, and, in case of further need, the district engineers' equipment and supplies are co-ordinated by the division engineers.

One of the most valuable aids that Pennsylvania has experienced in handling its snow problem is the co-operation of the Weather Bureau stations, from which each superintendent secures advance notice of coming storms, which enables each outfit to be in readiness for duty when the time comes.

Location.—In a number of cases, it has been possible to diminish the snow-removal work by giving particular attention to snow in the location and design of the improvements. By relocation of certain sections, laying the road on wind-swept ridges rather than against hillsides; in other cases by raising the grade line in cuts; and in places where it has not been possible to relocate or raise the grade by widening cuts and flaring slopes, conditions have been improved. All over the State, but particularly in the sections that are most troubled with drifting, a point has been made to remove, or have removed, unnecessary obstructions that would create drifting tendencies.

Snow Fence.—For a number of years Pennsylvania has recognized the value of wind-breaks to prevent drifting. The ideal way to protect a submerged piece of road would be by a tall dense hedge. Such wind-breaks may be practicable in the future, but probably are not so at present. Substitutes can best be furnished by portable snow fence.

At present, the most economical fence available is the wire and picket type. Standards and specifications should be drawn with a view to permitting open competition and encouraging improvement as well as describing minimum requirements. It would be difficult to show just how much money is saved by the use of snow fence, but the writer is confident that it is more economical to prevent drifts than to open them.

Equipment.—The problem of equipment is primarily in the choice between specially built snow-removal machines and tractors or trucks with adjustable

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1123.

plows. The trucks and tractors used for snow removal are busy during the construction season on maintenance operations.

The tractor rotary is typical of the special machinery. The adjustable plow is either V-type or mold-board, although recently the V-type has been improved and is generally in greater favor.

Expediency and economy are involved in the selection of equipment. A piece of equipment assigned to a section of road should be able to cope with all conditions on that section. All equipment should be charged to the work at sufficient rental to cover general charges, depreciation, interest, insurance, storage, etc., as well as actual operating costs; and if two or more types of equipment are capable of handling the work, that type which does the work at least cost, rental included, should be selected. Under some conditions special snow-removal equipment is desirable. For instance, the Pennsylvania Highway Department furnished rotary plows to the maintenance district that had the 137 in. of snowfall in the winter of 1925-26 and is satisfied that the greater cost was justified in keeping open roads that would have been blocked in spite of the efforts of less effective equipment. However, in other parts of the State where the fall was lighter, it was possible to keep the roads open and save money by using tractors and trucks fitted with adjustable plows.

It is not desirable to provide one piece of equipment for handling heavy drifts and other equipment for the light work between drifts. The amount of equipment provided is determined by the number of miles of road to be kept open and the number of miles each piece can handle. As Mr. Hamlin points out, there will be some variation in the length of the sections, but usually each piece of equipment will care for about 13 miles of road.

Operations.—Practical consideration sets a minimum limit of depth to about 2 in. for snow removal. Traffic is not seriously handicapped by less than a 2-in. depth and setting the plows closer to the pavement than this would increase the difficulty of operating. Accordingly, work customarily starts when the 2-in. depth is reached and continues as long as there is fall or drifting.

The roads should be kept open for two-way traffic if possible, but if storm conditions prevent this, a single track should be maintained open, with turn-outs at intervals for passing. At the expiration of the storm, this track can be widened.

When the storm has subsided and the roadways of the program mileage are open to two-way travel, the time has come to proceed more cautiously. Such equipment as betrays need of repairs should receive immediate attention. Some may be required to shove back snowbanks from the shoulders, to make room to dispose of additional snow in case of another storm. The other equipment may be scattered to open drifts on roads not on the program; and the caretakers and foremen, or patrolmen, on the program mileage, should proceed under supervision to open culverts and to cut shoulder drains through the snowbanks along the pavement. This clean-up work is essential if the successive snow removal operations are not to be impeded by the accumulation of preceding storms and if, when the thaws come, the snow water is to be removed without damage to the road.

Personnel.—The personnel feature of snow removal appears to be not a problem but rather the solution of a problem. One of the most serious difficulties in road maintenance has been rebuilding the organization in the field after winter suspension of activities. Snow-removal work furnishes winter employment, and thus holds a number of the best maintenance men.

Made up of picked men from maintenance forces, the snow crews exhibit a high morale. Comparatively new as the snow-removal activity is, they have firmly established for it the tradition of efficient and unflinching service.

Costs.—Costs will naturally vary with conditions. The method of accounting used, whether depreciation and interest on equipment are included, for instance, will also affect the figures. In making any comparison, these things must be taken into account.

An analysis of snow costs in Pennsylvania for the winter of 1925-26 (Table 2) is given with sufficient notes so that possibly parts of the data can be used for comparison in cases where the totals are not on equal basis.

TABLE 2.—ANALYSIS OF SNOW REMOVAL COSTS IN PENNSYLVANIA FOR THE WINTER OF 1925-26.

Items.	Program only.
Snowfall (U. S. Weather Bureau Report), in inches*	50.4
Snow fence,† in miles	101.0
Road, ‡ in miles	5 061
Product of miles and snowfall, in inch-miles	225 074
Cost of cinders and erecting, dismantling, and storage of snow fence per mile of road	$15.95
Reported snow removal,§ in inch-miles	357 138
Percentage of excess of reported snow removal (due to drifting), in inch-miles	40
Cost of snow removal per mile of road	$109.09
Cost of snow removal per mile of road per inch of snowfall	$2.16
Cost of snow removal per inch-mile of reported snow removal	$1.55

* State average.

† Approximately 91 miles of picket and wire, and 10 miles of "railroad" types.

‡ Mileage on program kept continuously open.

§ An inch-mile is the quantity of snow of 1-in. depth, 24-ft. width, and 1-mile length.

FINISHING AND CURING OF CONCRETE ROADS

Discussion*

By E. S. Borgquist, M. Am. Soc. C. E.

E. S. Borgquist,† M. Am. Soc. C. E. (by letter).‡—Perhaps no concrete is subjected to more severe usage than a road slab. Besides straight compression and frequent flexure stresses across weak spots in the sub-grade, there are heavy impact stresses, together with a continual abrasive action of different kinds of tires. Necessarily it takes a high quality of material to resist these forces and for this reason highway builders have been eager to adopt the best methods available for mixing, placing, and curing of concrete.

The State of California has built a great many more miles of concrete than any other Western State, and it is impressive to note the magnitude of the tests and the care with which they are carried out. The State of Utah has built more than 200 miles of concrete roads and has tried to keep abreast of the best practice.

A statement of mix in terms of 1:2:3 or a 1:2:4 is only relative. The Utah State Road Laboratory has proved that it is entirely practicable to design a mix after the method developed by D. A. Abrams, M. Am. Soc. C. E., by taking into account the grading of the materials used as expressed in his "fineness modulus".

An inspector of ordinary practical ability can readily be taught to screen gravel and sand and so compute the fineness modulus of each. The sand to be used is found by Professor Abrams' formula:

$$p = \frac{A - B}{A - C}(100)$$

in which,

P = the percentage of sand to be used in a given mix;
A = the fineness modulus of the gravel;
C = the fineness modulus of the sand; and
B = the fineness modulus of the mixed aggregate.§

In road work this is no longer a purely academic formula for laboratory processes but a thoroughly workable method for determining how much sand to use in the field. By means of this formula the writer has found that, varying the sand content in concrete according to the grading of the sand and

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* This discussion (of the paper by C. L. McKesson, Assoc. M. Am. Soc. C. E., presented at the meeting of the Highway Division, at Salt Lake City, Utah, on July 8, 1925, and published in August, 1926, *Proceedings*), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Asst. Prof. of Civ. Eng., Univ. of Arizona, Tucson, Ariz.

‡ Received by the Secretary, September 4, 1926.

§ *Bulletin No. 1*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

gravel, the mix was just as workable with 23% sand as with 50%, because the sand in the latter case was very coarse, in fact, almost a pea gravel.

In regard to water content, the State road engineers have found that they can work with a much dryer mix than they ever thought possible. If, as proved by Professor Abrams, a concrete more than twice as strong results from simply cutting down the percentage of water from 1.0 to 0.5 measured in volume of water as compared with volume of cement, surely it is worth trying to obtain the additional strength, particularly for road slab concrete. The 0.5 coefficient gives a concrete too dry for anything except laboratory test work, but it is possible to cut down the percentage of water to 0.7 and still obtain an entirely workable mix, if striking and finishing of the road slab is done with a finishing machine.

On a 10½-mile concrete job poured in 1922, on which the writer served as Resident Engineer, a 1:5 mix was used and a test block was made every day from the concrete dumped on the sub-grade, with the result that the whole season's work showed an average compressive strength of 4 300 lb. per sq. in. for 30-day concrete. Of course, only good clean gravel and washed sand were used.

Relative to expansion joints, Mr. McKesson states* that it is California practice to place a joint every 30 or 40 ft., regardless. In Utah, the practice is to place expansion joints only at the end of the forenoon run and of the afternoon run, thus giving two joints to be placed each day. It is difficult to obtain a "perfect" joint. The finishers usually build the joint too high so that there is a definite jar to a car riding over it. After the concrete has cured, the road will crack transversely every 30 or 40 ft., giving a contraction joint which opens at night and closes again on hot days. The joints thus formed by "Nature" occur only as needed. They are at least smooth to ride over and are quite as easily maintained as the expansion joints regularly placed square across the road at intervals of 30 or 40 ft. In Utah, one ⅜-in. elastite joint filler is used for each 25 lin. ft. poured, with a maximum of four such fillers placed side by side, thus giving a joint 1½ in. wide.

In the matter of curing, the State favors the method of "diking". In the West it is a difficult matter to get a contractor to keep the concrete damp if it is simply covered with a blanket of earth. If dikes are built and water is placed at least 3 in. deep on the slab, the concrete is much more likely to be properly cured and if it gets dry that fact is easily detected. Only in exceptional cases of scarcity of diking material is the sprinkling method allowed.

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1127.

EFFECT OF LIGHTNING ON A REINFORCED CONCRETE PAVEMENT

Discussion*

By T. A. Ross, M. Am. Soc. C. E.

T. A. Ross,† M. Am. Soc. C. E. (by letter).‡—Following the author's description of the make-up of the reinforced concrete pavement in question and its natural condition—in effect, a series of mutually isolated metal screens laid along the ground surface, but electrically insulated from it by dry concrete and sand, the latter of considerable depth—it appears that here is something closely analogous to a power transmission line erected with a series of short gaps, at 30-ft. intervals, in its conductors and these, in turn, supported on an imperfect type of insulator, in this case a silicious sand of indefinite moisture content. While silicious sand is itself a fairly good insulator any moisture present in it will lower its insulating properties.

The behavior of power transmission lines under lightning conditions—the term "conditions" is used advisedly—has been closely studied in recent years, and some of the knowledge thus gained can be applied to this case. A cloud that has become charged to a high electrical potential, or pressure, is possessed of a magnetic field which creates, by induction, a potential of varying intensity in any metallic or other conducting body that comes within the influence of the field. As long as the cloud remains charged and floats in the neighborhood, there will be an electrical equilibrium between it and the conductor which it has influenced. As the cloud, still undischarged, floats away, the charge induced by it in the conductor dies down in the latter to normal earth potential, without giving any evidence of its having been present. Should the conductor in question happen to be a power line in operation, there will likewise be no evidence of its presence on the line, or any disturbance whatever of normal working conditions, as long as the charge remains "bound" by the presence of the undischarged cloud.

Should, however, the cloud discharge itself close by, having approached so near to some "earthed" body or conductor that its potential is able to break down the intervening air, then, at this same instant of discharge, the already charged conductor finds itself left in a state of high electrical tension which may be of the order of many hundreds of thousands of volts. No practical commercial insulation—let alone the fortuitous insulation supplied to a metal reinforcement by dry concrete and sand—can withstand such a pressure, and the conductor discharges itself to earth simultaneously with the cloud.

* This discussion (of the paper by Winston E. Wheat, Assoc. M. Am. Soc. C. E., published in September, 1926, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Lt.-Col., R. E. (*Retired*); Care, The Foundation Co., New York, N. Y.

‡ Received by the Secretary, October 6, 1926.

The power engineer provides for the contingency by connecting expensive but efficient lightning arresters to his apparatus. What is the highway engineer to do? It is submitted that he need do nothing. Under all normal conditions he will find reinforced concrete pavements so efficiently—or at least sufficiently—connected to and incorporated with the main body of earth that there will not be the least likelihood of their becoming charged to a higher potential than that of the earth. If a cloud should happen to discharge itself directly over a point on the highway that is a dispensation of Providence for which he cannot be expected to provide in his specifications. It is happening probably every day in the year without any one being aware of it, or of any harm resulting. Many construction engineers will recall having dug up so-called "fulgurites", particularly in sandy soil. These fused cores of earth—in appearance not unlike some stalactites—have been made by the lightning discharge as it passed down to the moister strata of the lower earth levels and there dissipated its energy.

Referring to the "Gulf Beach Highway", from the evidence presented by the author, the writer would deduce that the section of damaged roadway had become charged above earth potential in the manner described. Mr. Wheat does not give any direct evidence that the pavement was actually struck by a lightning discharge—the writer prefers the latter term to "bolt of lightning"—nor is it necessary that it should have been in order to account for the damage done.

What probably happened was that the charged sections discharged themselves simultaneously with a lightning discharge in the immediate vicinity. The pavement was obviously not a good "earth", but rather a well-insulated body, and, as such, lightning would not tend to discharge directly on it; on the other hand, it was itself in good condition to become charged prior to the lightning discharge.

The author states that the incident occurred during a thunderstorm. It is rather a pity that he did not say explicitly whether rain was falling at the time, or shortly before, because both are material. If one can assume either, an explanation is then available for the damage at the expansion joints, because there it would be most likely for moisture to penetrate and descend into the sub-grade. In any event moister conditions would prevail under these joints than under the slab. In speaking of moisture, the term in this case is used relatively—an electrical potential of the order under consideration will discharge itself through moisture which only a chemical test would reveal, and through many feet of air as well—and the author's statement that moisture is essential for grounding lightning discharges must be read in that light.

Each section would discharge itself through the point of least resistance to earth—in this case the joints. The accompanying flash—in effect, a lightning discharge in miniature—would create steam and gas to disrupt material and heat to set fire to the asphalt filler.

The peculiarity that the concrete was usually broken at the eastern end only of each slab could be explained by the fact that the reinforcement approached more closely to that end, and thereby provided a shorter distance to earth; a contributing cause would be that if the pavement was laid on a down grade from west to east, the surface water in flowing off

each slab toward its lower, or eastern, end would tend to seep through the upper side of each expansion joint. In that case the high end of each slab would be the dryer and the low end the damper and, therefore, the better conductor to earth.

That the damage appeared on alternate sides of the pavement can also be explained by the reinforcement having been laid with its ends nearer the surface on alternate corners of each slab. In any case the writer would look for simple mechanical reasons, such as comparative measurements, to explain these phenomena in preference to searching for obscure electrical ones.

In spite of all precautions lightning will sometimes defeat the power engineer. An insulated conductor—be it a power transmission line or a "Gulf Beach Highway"—once it has become charged by a high-potential cloud may be likened to a long pipe, open at both ends, and charged in the middle with dynamite. If the dynamite explodes it will wreak its damage in its immediate neighborhood—the open ends of the pipe are not going to help much. Like dynamite, analogies are sometimes dangerous, and this one must not be pushed too far. It will help one to realize, however, that a charged body will probably free itself of its "bound" charge over the shortest path to earth and without much consideration of the insulating properties of that path. Thus, the power engineer may find many of his line insulators broken down by a lightning discharge without either his line being directly struck or his lightning arresters failing to operate. Incidentally, cases have occurred where power lines out of use and disconnected have yet broken down their insulators during a thunderstorm.

Similarly, it is not going to help much—even if it were practicable—to make pavement reinforcement electrically and mechanically continuous, without impairing the free expansion of the pavement, and then providing it with special connections to earth. As has been already pointed out, it is and always normally will be efficiently earthed by its very nature, and no possibility will ever arise of it being charged to a higher potential than the general body of the surrounding earth.

The circumstances and conditions described in the paper are so rarely met that it is probably better economy to face the cost of occasional small repairs rather than attempt to provide costly, and not always certain, protection against lightning for a pavement which is to be laid on very dry sand or earth. The danger to life is practically non-existent; it is certainly less than the normal risks involved in using the road for travel.

Steel-frame buildings, bridges, railroad and street railway tracks, etc., are by their very nature effectively earthed; they cannot acquire a "bound" charge and can accept direct lightning discharges with impunity. This does not mean that certain types of structures should not be specially protected against lightning discharges and lightning conditions. Powder magazines and power transmission lines, for example, require special study for their own peculiar conditions on account of the extra risks involved. Neither is it enough to provide special protection for them; the protection equipment should be efficiently maintained, and tested at regular intervals by expert engineers.

SAG-TENSION CURVES

Discussion*

By Messrs. W. Marshall Page and J. C. Stevens

W. Marshall Page,† Esq. (by letter).‡—Mr. Parker's suggestion of using a "factor length" instead of the usual span length in the parabola formula, and of making provision for the tension at the support rather than using the tension at the low point of the span, is a decided improvement. The increase in accuracy obtained makes this method with his modifications equivalent to the catenary methods.

For short span construction, the ordinary parabola method has been as satisfactory as the various catenary methods. It was, however, subject to some criticism in the calculation of sags for long spans, as these showed a greater safety than actually existed, or gave a stringing sag less than that obtained by other methods. The error tended to decrease the actual factor of safety. Probably one of the main objections to the ordinary parabola method was the uncertainty regarding this error.

It is true that many of the other factors that enter into a sag problem are also subject to considerable variation, but these have been generally understood or accepted, and have been provided for by using conservative assumptions. The incidental errors were usually in such a direction that the actual factor of safety was increased. For example, cables are often 10% stronger than the rated strength used in the design calculations.

Mr. Parker gives a very clear demonstration of the parabolic method for the solution of sag problems, and cites the advantages of this method. The basic formula is simple and easily handled. The graphic solutions as given in Fig. 1§ definitely show the relation between the sag and tension of a suspended wire under various conditions of loading. This is a very desirable feature for those who only occasionally have sag problems.

The use of the "factor length" and the actual tension at the support should be adopted by all who are using the parabola method. It retains all the advantages of the parabola and the increase in accuracy makes it applicable for the calculations of spans of all lengths.

J. C. Stevens,‖ M. Am. Soc. C. E. (by letter).¶—The author presents a method both simple and practical for the solution of the sag of cable spans.

* This discussion (of the paper by Edwin S. Parker, Assoc. M. Am. Soc. C. E., published in September, 1926, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Engr., Copperweld Steel Co., New York, N. Y.

‡ Received by the Secretary, October 18, 1926.

§ *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1338.

‖ Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

¶ Received by the Secretary, October 18, 1926.

A number of years ago the writer prepared a diagram for the erection sags to be given cables used in supporting cars for stream-measurement work.* The requirements in that work principally hinged about the fact that the operator must be in no danger of touching the water when the stream was in flood.

It is often impracticable to give the cable a predetermined tension, and the writer outlined a method whereby the required tension at erection could be given by an "erection sag," which would be inversely proportional to the tension. It is a simple matter to draw a cable up to a given sag, but not always so easy actually to measure the tension.

One difficulty encountered by the writer—and the author has thrown no light on it—was to correct for the effect of temperature. It is simple once the linear coefficient of expansion is known. The writer questions whether the coefficient for the material of which the cable is composed will apply to the cable itself. He recently requested the U. S. Bureau of Standards to forward him data on this question, but that Bureau stated it had no information on the subject.

It appears that the coefficient of linear expansion of a cable as compared with a wire of the same material must vary with the lay of the cable and also with the tension it is under at the time. If the author has any data on this point he will render the profession a service by publishing them.

* *Engineering News*, May 6, 1909, p. 483, and July 15, 1909, p. 80.

THE ENGINEER AND THE TOWN PLAN

Discussion*

By Charles H. Paul, M. Am. Soc. C. E.

Charles H. Paul,† M. Am. Soc. C. E.—In the work of city planning, the engineer is given an opportunity to get in close touch with the public, such as is not offered in many other branches of engineering. The speaker has had the privilege of serving on the City Plan Board in Dayton, Ohio, and that work has resulted in the fullest co-operation between the City Plan Board, the Real Estate Board, the Chamber of Commerce, and various other business men's organizations. Recognition of the need of engineering service on the Plan Board was shown by the fact that two of the seven members appointed were engineers, and one other was a plant executive with a definitely engineering turn of mind. There is no question that these city planning studies are bringing to the minds of the public the value of engineering training and engineering analysis in the solution of many of the problems which are faced by the municipality.

One of the problems that this Plan Board has recently worked out is a serious grade-crossing situation. For the last fifteen years elimination of grade crossings in Dayton has been discussed. Various solutions have been offered, but the situation has never been studied in a manner that would give full consideration and weight to the advantages and disadvantages of the various possible solutions. The City Plan Board has had the problem analyzed in that way, and has taken into account, and made a part of the same problem, the re-arrangement of streets and thoroughfares in the vicinity of the tracks, the freight stations, and the passenger station. Although the cost of grade-crossing elimination is bound to increase year by year as the actual work is postponed, it has been remarked publicly by one of the city officials that the delay and increased cost, in this case, are well worth while because the present solution is so much better than any that had been offered before.

The city planning engineer, just by the nature of his work, is dealing directly with the public most of the time. He has the opportunity to broaden and develop himself because of this direct contact with the various public groups, and, at the same time, he has the chance to show the public, both as individuals and as groups, that engineering training and the engineering approach to a municipal problem usually results in the most direct, satisfactory, and economical solution. Contributions of this kind by engineers to civic work will do more than almost anything else to put the profession in its proper place in the eyes of the public.

* Discussion on the paper by James Ewing, Esq., continued from November, 1926, *Proceedings*.

† Managing-Director, Dayton Industrial Assoc.; Cons. Engr., Dayton, Ohio.

AERIAL SURVEYS FOR CITY PLANNING

Discussion*

BY MESSRS. ARTHUR S. TUTTLE, F. L. OLMSTED, AND CHARLES N. GREEN.

ARTHUR S. TUTTLE,† M. AM. SOC. C. E. (by letter).‡—This paper appears to cover the subject so thoroughly as to leave little opportunity for further amplification or discussion. It may be advisable, however, to emphasize a number of points to which the author refers and to explain a little more fully the reasons that determined the selection of the 600-ft. scale for the New York City air map.

While the question of cost was, of course, an important consideration, the prime factors in the determination of the scale to be adopted were the utility and convenience of the completed map. The only official map of the entire city, completed in 1915 and revised systematically to date to show all adopted map changes, is on a scale of 600 ft. to 1 in. and comprises 35 sheets or sections, each measuring 29 by 42½ in., including a 1-in. lap. This scale has been demonstrated by long experience to meet all the requirements of a general map and to serve as an index of the large scale plans on which the details of each individual map change or project are shown. It seemed obvious that the usefulness of an air map in the visualization of city problems would be greatly enhanced if it could be studied in connection with a line map of the corresponding area, and that for this purpose there would be a distinct advantage in having the scales of the two maps identical. In the writer's opinion, this contention, which largely determined the adoption of the 600-ft. scale, has been conclusively demonstrated to have been sound, and in view of the size of the area covered it does not appear that any advantage would have resulted from using a larger scale.

The impracticability of reproducing or handling photographic copies of the air map, measuring 29 by 42½ in. in size, led to the plan of subdividing the area covered by each section of the city map into four sub-sections, each bearing the same serial number as the corresponding map section, to which was added a distinguishing letter to complete the identification of the individual photographic map sections. It may be of interest to state that the mounted sectional photographic map sheets make an atlas measuring 2½ in. in thickness, without covers.

* This discussion (of the paper by Gerard H. Matthes, M. Am. Soc. C. E., presented at the meeting of City Planning Division, Montreal, Que., Canada, October 15, 1926, and published in September, 1926, *Proceedings*), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

‡ Received by the Secretary, October 25, 1926.

With regard to accuracy, the contract under which the New York City air map was furnished provided that measurements made between any two points on the map not less than 9 in. (5 400 ft.) apart should not vary more than 3% from the corresponding distance on the control system. In this case the control was the city map previously referred to, the groundwork of which is based on a precise triangulation survey executed between 1903 and 1908 jointly by the U. S. Coast and Geodetic Survey and the City of New York. Before the air map was accepted it was seen that this provision of the contract had been rigidly complied with and a more recent test involving the comparison of sixteen scaled distances, ranging from 5 500 ft. to about 6 400 ft. in length, has indicated a maximum discrepancy of only 1.6% and an average of 0.6 per cent.

In discussing the question of cost the author notes the requirements relating to special appliances and highly trained personnel and calls attention to the fact that the taking of photographs is limited to perfectly clear days which, he states, occur on an average of 1 day in 7, or about 15% of the time, for the greater part of the United States. In partial confirmation of this condition it may be stated that between August 9 and October 17, 1923, when the New York City area was photographed, there were only 11 days when satisfactory atmospheric conditions prevailed and that it was subsequently found necessary to re-photograph a part of the Manhattan area to secure clearer definition. It should also be remembered that photographs must be secured toward the middle of the day in order to minimize the effect of long shadows, and that for heavily wooded areas the topographical features can only be secured at a time of the year when the foliage is not dense. The last consideration will not generally apply in the case of city work.

A carefully made air map is of inestimable value as an adjunct to the visualization and solution of city planning problems. In some instances where other maps or data are lacking and the time element is the all-important consideration, photography supplies a means which has hitherto been lacking for securing fairly accurate and detailed information with the greatest possible dispatch and economy. In general, however, such maps made entirely by photographic processes will not supplant the draftsman's portrayal of the situation, based on accurate and detailed ground surveys, but will be used rather to supplement and clarify the picture.

F. L. Olmsted,* Esq. (by letter).†—The writer can only confirm in general all that Mr. Matthes has said regarding aerial surveys for city planning, especially his statement of the advantages of using the original aerial photographs in connection with a line map.

On one occasion recently the writer had made arrangements for the preparation of a controlled mosaic map covering about 250 sq. miles, as a basis for a preliminary general city plan, but owing to bad weather and other adverse conditions only the original unadjusted aerial photographs were ready at the time it became necessary to push the field studies for the plan. The mosaic was to have been enlarged about two and one-half diameters from the scale

* Olmsted Brothers, Landscape Archts., Brookline, Mass.

† Received by the Secretary, October 28, 1926.

of the original photographs and to be on the scale of 400 ft. to 1 in. The photographs, varying considerably in individual scale but averaging about 1 000 ft. to 1 in., were matched together in long strips of overlapping prints fastened by adhesive paper "hinges". Many section corners and section lines were recognizable by the streets, roads, and fence lines. Large sheets of tracing cloth were ruled off into mile squares at the scale of 1 000 ft. to 1 in., to represent the approximate locations of section lines, although these were known, of course, not to be exactly mile squares. With these as a rough control a hasty tracing was made from the photographic strips showing the main features—roads, ponds, streams, etc.—as a general rough guide map.

Field studies were then made using the strips of original photographs, folded accordion fashion and carried on a strip of "compo-board" held by rubber bands, prints of the general guide-map tracing being carried along for reference. Field notes were made in ink and in pencil directly on the photographic prints, the wealth of detail—trees, fences, buildings, roads, trails, ponds, and streams—sufficing to locate on the prints instantly and with precision (in relation to surrounding objects) any point observed on the ground. Elevations were spotted on the photographs at the principal points along lines where vertical control had been run by field parties. The variations in scale between adjacent photographs of course made it impossible to record without danger of considerable distortion alignments extending across more than one photograph and not represented on the ground by visible objects, such as roads or fences, and the map was wholly without contours.

As nearly all the topography was very gentle and the lines of natural drainage were well indicated by brooks and ditches, it was possible, subject to later adjustment of alignments of proposed streets extending across the matching lines of the photographs, to complete nearly all the field work for the general plan before making either a "controller" mosaic map or a proper controlled line map. The problem of subsequently transferring and adjusting the details of the general plan to the completed controlled map at uniform scale presented no serious difficulty, as the plan was practically complete in detail and closely tied at all essential points to recognizable objects, with a local error kept within a limit of less than 20 ft. wherever that degree of precision was important.

This "cart-before-the-horse" method should be adopted only where considerations of speed absolutely necessitate it; but the writer's experience with it strongly emphasizes the advantages of working directly on the original aerial photographs with their complete detail, in conjunction with a line map based on those photographs and showing only a small fraction of their detail but correct in scale. The occasional use of a magnifying glass, and, for certain purposes, of a stereoscope, with these small-scale photographs and the taking of some pains in drafting and note-making, permit the accomplishment of better results in less time than is possible by ordinary methods with maps on a much larger scale. If time and funds permit, it is easier to work such plans out on maps at a scale of 400 ft. to 1 in.; but by taking the necessary pains in the planning work equally good results can be produced at a greatly reduced

total cost for mapping and planning if done at a smaller scale. However, 1 000 ft. to 1 in. is near the practicable workable limit.

Issue can be taken with Mr. Matthes on only one point. He is quite right in stating that horizontal control of a geodetic precision is wholly unnecessary for maps intended as a basis for general city planning; but it is equally true, as he also indicates, that such general plans must be followed in course of time with large-scale precise fragmentary plans for construction, and for cadastral purposes. In the long run, also, it saves a great deal of trouble for the city's engineering department and for landowners if the principal monuments to which both fragmentary cadastral surveys and the city's fragmentary detailed construction surveys must be tied, are located once for all in relation to a single uniform system of co-ordinates by means of a control survey of a high degree of precision. The making of a general map of the whole territory, even if its scale is to be so small that a non-cumulative error of 4 ft. is negligible, necessitates some general system of horizontal and vertical control for the whole territory and makes an occasion for getting a main skeleton of primary control sufficiently precise to be used for all subsequent detailed surveys on any scale, so that as they are gradually extended they all will fit without "fudging" where they meet and overlap.

Charles N. Green,* M. Am. Soc. C. E.—In a study of Northern New Jersey with the idea of betterment of transit facilities, air pictures were made of an area of about 150 sq. miles. From east to west the area extended approximately from the Hudson River to the Orange Mountains, and from north to south from Paterson to and including Newark. All the photographs were made oblique, and were taken at an elevation of about 10 000 ft. The photographs, of course, were not in detail. If a particular area were required in detail, as, for example, that point where the Erie and Lackawanna Railroads cross before entering the tunnels in Jersey City, and also some entire towns, the prints were enlarged.

That particular point—the Erie-Lackawanna crossing—is a most difficult spot to photograph. Apparently, it can only be done on Sunday morning with a northwest wind. This crossing, not at grade, with more than fifty trains passing over the two roads on a week-day morning, together with moisture in the air, bring about an atmospheric condition so dense that it cannot be penetrated even with a screen. Several attempts were made and none of them has been satisfactory.

About 40% of each photograph is all that can be used; the remainder has to be covered by the overlap on the next one. The photographs were taken in long sweeps across the area, probably one every mile or two so that the area was fairly well covered. This method, however, requires a location map to show approximately the area covered by each photograph. With this ready means of reference, any area may be picked out and studied in detail, with the further aid of an enlargement if desired. For the purpose for which these photographs were made, they have been very satisfactory and have saved an incalculable amount of time.

* New York, N. Y.

GRIT CHAMBER PRACTICE

A SYMPOSIUM

Discussion*

By Harrison P. Eddy, M. Am. Soc. C. E.

Harrison P. Eddy,† M. Am. Soc. C. E.—The subject of grit chambers, as pointed out by Mr. Macallum,‡ is one that has not been studied thoroughly in all cases. Sometimes, apparently, such chambers have been installed because the practice had been adopted elsewhere.

There have been many instances, however, where grit chambers have proved of value. At Worcester, Mass., about twenty years ago, they were installed as a means of preventing the large expenditure required for labor in the removal of heavy sand and gravel deposits from the sedimentation tanks. At that time the sewage of Worcester was treated by chemical precipitation.

At Fitchburg, Mass., where the topography of the city is very rugged and where there are many miles of gravel and cinder roads from which large quantities of sand and detritus reach the sewers, grit chambers have been installed at the up-stream end of a long inverted siphon leading to the treatment plant. It is of interest to note that in spite of this it has been necessary to clean the lower compartments of the Imhoff tanks at this plant. In these compartments accumulations of inert material consisting in part of sand, gravel, and cinders gradually reduce the total effective capacity for sludge digestion until the material is removed. At Fitchburg, designs have been prepared for an additional set of grit chambers, in order to reduce the frequency of cleaning the sludge compartments.

There has recently developed in the United States another problem which has rather overshadowed that of grit handling. This is the removal of oil. Oil comes from various places such as automobile service stations, factories, and gas-works. At Akron, Ohio, as much as 300 gal. of heavy oil have been skimmed from the surface of existing Imhoff tanks in a single day, even though the plant was receiving only a moderate portion of the entire sewage flow. Similar conditions have occurred at Schenectady, N. Y., Milwaukee, Wis., Chicago, Ill., and Worcester; and, at Detroit, Mich., large quantities of oil flow from the sewers into the river.

In order to remove this oil promptly and inoffensively and at the same time remove the grit and coarser suspended solids, a skimming detritus tank

* Discussion on the Symposium on Grit Chamber Practice continued from November, 1926, *Proceedings*.

† Cons. Engr. (Metcalf & Eddy), Boston, Mass.

‡ *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1867.

has been devised and is now under construction at Akron in connection with its new sewage treatment plant. The detritus plant comprises two round tanks, 55 ft. in diameter and 13 ft. deep. The influent will be admitted to one side of the tanks and the effluent will pass out through submerged orifices. For average ultimate rates of flow these tanks will provide a detention of 15 min.

The tanks will be equipped with adjustable weirs and revolving arms, adjusted so as to skim the grease and cause it to flow on to small drying or absorbing beds. A part of the suspended solids, including the grit, will settle. Revolving plows will be provided to assist the passage of the settled solids to a central sump in each tank. These sumps will be connected with centrifugal pumps, which will draw the sludge and grit together with approximately 20% of the sewage from the bottom of the detritus tanks.

This concentrated sewage will flow through one of a pair of grit chambers. Each grit chamber will be about 3 ft. wide and have an effective depth of flow of 3.5 ft. Each chamber will be provided with a flight conveyor to scrape the grit from the chamber up an incline and discharge it into a storage chamber, from which it will be dropped into cars and thus carried to the fill.

The flow from the grit chambers will contain considerable suspended organic matter for the removal of the coarser portion of which two mechanical fine screens with $\frac{1}{8}$-in. slots, will be provided. These screens will be 6 ft. long by 6 ft. 6 in. in diameter. The screenings will be disposed of either by incineration or by use for filling.

The general plan of providing detritus tanks and accessory equipment has several advantages. Such an installation will prevent relatively large quantities of oil, grease, and floating solids from accumulating on the surface of the sewage in the Imhoff tanks and rendering them unsightly or expensive to maintain in a presentable condition. The detritus tanks will be at least as effective as grit chambers and fine screens in removing grit and the coarser suspended solids. This removal of the coarser suspended matter will reduce the danger of trouble with scum in the Imhoff tanks. In the case of Akron it is estimated that such detritus tanks for the entire sewage flow, with grit chambers and fine screens for the concentrated sewage equivalent to 20% of the total flow, will result in a substantial saving in the construction and operation costs over similar costs for grit chambers and fine screens for the total flow.

WATER-RATIO SPECIFICATION FOR CONCRETE

Discussion*

By Messrs. T. P. Watson and Charles S. Whitney.

T. P. Watson,† M. Am. Soc. C. E. (by letter).‡—It is the writer's opinion, in agreement with some authorities, that the quality of concrete is dependent principally on the quality of the mortar, or the combination of water, cement, and fine aggregates. He does not agree with the authors in their hypothesis§ that "* * * the fundamental principle that the strength and other desirable properties of concrete are definitely determined by the proportion of the water to the cement in the mixture, provided only that the concrete is plastic and workable and the aggregates are clean, durable, and structurally sound."

This theory is not substantiated by the data presented. A comparison of the results in Table 1‖ with those represented in Fig. 1,¶ discloses differences that are typical of the inherent futility of a concrete strength specification for general application because of the different strengths obtained by the use of different cements and different types of aggregates, and on account of the personal equation in the making and curing of test specimens. The data contained in Table 4 were taken from Table 1.

TABLE 4.—Compressive Strengths of Concrete for Various Water Ratios.

Mixture by volume, in terms of dry rodded volumes of aggregates.	Quantities of Mixing Water (U. S. Gallons) per Sack (94 Lb.) of Cement.				
	6.	6½.	7.	7½.	8.
1:1.5:0	4 300	3 970	.....	.....	....
1:2 :0	4 090	3 740	3 340	.....	....
1:2.5:0	3 560	3 510	3 430	3 210	....
1:3 :0	.....	.....	.....	2 980	....

It will be noted from Table 4 that there was a general decrease in strengths as the proportions of fine aggregates were increased, even if the same water-cement ratios were maintained, and that when a proportion of 1:2.5 was reached, the water-cement ratios affected the strengths in a much lessened degree.

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* Discussion on the paper by F. R. McMillan, M. Am. Soc. C. E., and Stanton Walker, Assoc. M. Am. Soc. C. E., continued from November, 1926, *Proceedings*.

† Asst. Engr., Eng. Dept., P. R. R., Pittsburgh, Pa.

‡ Received by the Secretary, September 28, 1926.

§ *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1407.

‖ *Loc. cit.*, p. 1409.

¶ *Loc. cit.*, p. 1408.

Further analysis of Table 1 discloses that there was a definite relation between the maximum concrete strengths for the various water-cement ratios and particular proportions of fine and coarse aggregates. Also, for a given water ratio and fixed proportions of cement and fine aggregate, the introduction of a small quantity of coarse aggregate almost invariably decreased the strengths; and increased proportions of coarse aggregates produced higher strengths until proportions were reached at which the maximum strengths were obtained. Thereafter further increase of the proportion of coarse aggregate resulted in lower strengths.

The lower strengths obtained from mixtures containing the smaller proportions of coarse aggregates were caused by the introduction into the mortar of planes of cleavage along the surfaces of the particles of the coarse aggregates. The higher strengths obtained from mixtures containing larger proportions of coarse aggregates were caused by the absorption into the coarse aggregates of a part of the water and by the phenomena of the curing of the concrete. This was aided by the available moisture temporarily stored in the coarse aggregates, thereby increasing the mortar strengths sufficiently to overcome the structural weakness of the concrete caused by the additional planes of cleavage due to the increased proportions of coarse aggregates. When the proportions of coarse aggregates were increased beyond the point where the mortar filled the voids the strengths became less.

Compressive strength has answered a useful purpose in spurring concrete users to strive for the better properties of concrete; but compressive strength is far from the basic essential quality required for the vast majority of concrete structures. The quality most desired is sufficient durability to withstand the destructive agents to which the concrete will be exposed.

Lasting impermeability is probably the cardinal quality to be sought for durable concrete. Research regarding permeability usually proves that the least permeable concretes were made with comparatively fine sands, yet the paradoxical tendency of concrete authorities is to advocate the use of the coarser gradings of sands because of the higher strengths obtained.

The same reasoning may be used with gradations of fine aggregates as was used previously in referring to concrete strengths. The higher strengths obtained with the coarser gradings of fine aggregate are due to the absorption and phenomena of curing, and to the relatively rich matrix of those fine particles of the fine aggregate in combination with the water and cement, in which are suspended the larger particles erroneously assumed to be fine aggregate, but, in reality, when larger than some unknown size, found to be parts of coarse aggregate.

The question of the maximum permissible allowance of fine aggregate passing a No. 100 sieve is pertinent when considering durability and is a matter too frequently overlooked. Fine aggregates with any considerable percentage passing a No. 100 sieve should be particularly investigated. If no other aggregate is available a compensating increase in the proportion of cement should be made, because in the usual run of fine aggregates the proportion that passes a No. 100 sieve is practically an inert powder which acts as an adulterant to the cement and produces a less durable mortar.

It is generally conceded that concretes of very low strengths and other theoretically undesirable properties have given satisfactory service when not exposed to outside climatic conditions, frequent alternate wetting and drying, hydrostatic pressures, or other destructive agents. It would seem logical, therefore, that concrete for particular purposes should be specified in accordance with the conditions of exposure and durability.

The writer is in agreement with the authors that the water-cement ratio is a basis for concrete specifications. The principal purpose of the suggested amendments to the proposed specification* is as follows: (*a*) The avoidance of as many technical terms as possible; (*b*) the avoidance of a strength specification; (*c*) the fixing of definite proportions of water, cement, and fine aggregate; and (*d*) the sanction of the proper use of smaller gradings of fine aggregates. The following amendments to the proposed specification are suggested:

Omit in its entirety the Section entitled "Suggestions to Bidders".

Water-Cement Ratio.—Omit this Article and substitute the following:

"*Concrete.*—Concrete shall be placed in accordance with classifications indicated on the plans and the following proportions of water, cement, and fine aggregate (dependent on the size classification hereinafter described) shall be combined with such proportions of coarse aggregate as will produce a mixture that can be readily puddled in the forms and around reinforcement without segregation and that will insure surfaces free from honeycomb when forms are removed.

Concrete classification.	Proportions.					
	Water, in U. S. gallons.	Cement, in sacks (94 lb.).	* Fine aggregate, in cubic feet saturated.			
			Size classifications.			
			No. 4.	No. 8.	No. 16.	No. 30.
A	†5.0	1.0	1.3	1.3	1.0	0.9
B	†6.0	1.0	1.6	1.6	1.3	1.2
C	†7.0	1.0	2.0	2.0	1.7	1.5
D	†8.0	1.0	2.4	2.4	2.1	1.9

* If fine aggregate is measured in a batcher or other receptable by loose volume its loose volumetric relation to a saturated volume shall be determined.

† Moisture contained in the aggregates must be deducted.

"The method of measuring materials shall be such that all proportions are closely controlled and easily checked at any time.

"Aggregates shall be such as will insure uniform quality and gradings.†

"*Concrete Classifications.*—

"(*A*) Thin reinforced slab sections exposed to outside climatic conditions.

"(*B*) Reinforced beams or columns, thin reinforced walls, copings, projections, water-tables, or horizontal surfaces on which water is

* *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1418.

† With a definite quality of mortar the coarse aggregate acts as an economical filler and as long as the proportions are such that the mass is readily placed without segregation there will always be sufficient mortar to fill the voids.

likely to accumulate when exposed to outside climatic conditions or especial interior conditions.

"(C) Interior structural members; and structures or structural members not reinforced, with comparatively vertical surfaces, when exposed to outside climatic conditions.

"(D) Foundations not exposed to outside climatic conditions."

Measuring Moisture in the Aggregate.—This Article should be amended to restrict variance in moisture so as not to exceed 1 lb. in each 100 lb. of aggregate. The allowance of 2 lb. is excessive and would defeat the application of the water-cement theory.

A clause should be inserted to prevent the allowance of any additional water on account of the absorption by the coarse aggregate. Coarse aggregate of such porosity as will absorb an appreciable quantity of water should be surrounded by a mortar of a lower water-cement ratio to offset the permeability of this type of aggregate. If no allowance for absorption is made in the quantity of water the capillarity of this type of aggregate to some extent automatically accomplishes the desired result.

Concrete Proportions and Consistency.—This Section should be omitted. The personal equation entering into the making of slump tests is such that, in especially qualified hands, the slumps are only a guide to the water content, and exacting methods of proportioning are used to insure identical quantities of all proportions of uniformly graded fine and coarse aggregates. The slump test should be avoided as part of a specification.

Control of Proportions.—This Article should be omitted.

Tests of Concrete.—This Section should be omitted. Although it is desirable that frequent tests should be made to satisfy the Architect or Engineer of the uniformity of the concrete as accurately gauged by concrete specimens when properly made and cured, they are not pertinent to the specifications.

Fine Aggregate.—This paragraph should be amended by omitting the second sentence and substituting the following:

"Fine aggregate shall be within one of the following size classifications:

Size Classification.	To Pass.	Retained at Least.	
No. 4	3/8-in. sieve 100%	No. 8 sieve 15%	No. 100 sieve 97%
No. 8	No. 4 sieve 100%	No. 16 sieve 15%	No. 100 sieve 97%
No. 16	No. 8 sieve 100%	No. 30 sieve 15%	No. 100 sieve 97%
No. 30	No. 16 sieve 100%	No. 50 sieve 15%	No. 100 sieve 97%"

Mixing Concrete.—This Section should be amended to increase the minimum mixing interval to 1½ min.

Depositing Concrete.—The third paragraph of this Article should be amended to limit temperatures of not less than 50° Fahr., or more than 90° Fahr.

Experience has shown that under certain seasonal conditions the placing of concrete at a temperature as low as 40° Fahr. involves unnecessary risk. The maximum temperature of the concrete should not be above 90° because when the concrete temperature is higher there is danger of structural complications from too hasty setting. The maintenance of a temperature of about 70° Fahr. will give very satisfactory results.

CHARLES S. WHITNEY,* M. AM. SOC. C. E. (by letter).†—The writer is not yet convinced of the necessity or advisability of the use of a specification for concrete that does not give the contractor definite proportions on which to base his estimate of cost. While the method of technical control of proportioning may be explained in the specification so that the contractor will know what to expect, the administration of it should be entirely in the hands of the engineer. The purpose of any specification should be to insure to the owner a good job and to the contractor a reasonable compensation. If the control of the proportioning is under the direction of the engineer employed by the owner, the saving effected by such control should go to the owner. In other words, the owner should pay a reasonable price for the materials and labor, and the contractor should be relieved in so far as possible of the gamble which results from an indefinite specification. If competition is desired, it is fair that the contractors should estimate on the same quantities, so that the contract will not go to the one who happens to be the most optimistic about the strength of his concrete.

For uniform bidding, the specification should give definite instructions as to the quality of materials and the proportions to be used. It may then be provided that the engineer may establish the exact proportions by any desirable method after samples of the materials have been submitted and during construction, and that the owner is to be credited or charged with the difference in cost of the materials. It is also well to specify as accurately as possible the consistency desired, that there may be no misunderstanding. Such a specification leaves the engineer entirely free to proportion the concrete in any way he may think best without depriving the contractor of any of his profit and makes possible the most efficient use of the materials and of the owner's funds.

If the strength of concrete depended entirely on the water cement ratio, the selection of aggregate would be very easy, but the problem is not as simple as that. The only disadvantage of the form of specification suggested by the writer is that the owner will not know in advance exactly what the total cost will be. It should be possible, however, for the engineer to predict the proper proportions closely enough, so that the change made during construction will have little effect on the cost. The owner will pay for only what he receives, and the contractor will not have a financial interest in using the cheapest possible mix. In case the owner is not represented by an independent engineer and the proportioning is in the hands of the contractor's force, it would seem desirable to include in the specification some form of strength clause. Even then the payment could be based on the actual proportions used.

* Cons. Engr., Milwaukee, Wis.

† Received by the Secretary, October 15, 1926.

UNIT STRESSES IN STRUCTURAL MATERIALS
A SYMPOSIUM

Discussion*

BY MESSRS. G. L. TAYLOR, LEWIS D. RIGHTS, RUDOLPH P. MILLER, L. S. MOISSEIFF, GUSTAV LINDENTHAL, ROBERT C. STRACHAN, AND CHARLES S. WHITNEY.

G. L. TAYLOR,† M. AM. SOC. C. E.—Attention should be called to the fact that the report‡ of the Joint Committee on Specifications for Concrete and Reinforced Concrete has been incorporated in the report of the Building Code Committee of the U. S. Department of Commerce practically as a whole; but in the case of the section dealing with steel, practically a new specification has been written. This new specification does not differ materially from the specifications of the American Institute of Steel Construction, but still it is different.

Professor Turneaure called attention to the limitation of the compressive stress for columns.§ The recommendations of the Building Code Committee allow an increase of 25% when wind stresses are included, whereas the specifications of the American Institute of Steel Construction allow 33⅓ per cent.

One of the chief aims of the Building Code Committee is to secure standard practice, and it would appear, as Professor Turneaure also pointed out, that the stresses for steel have been made more conservative than those for reinforced concrete. It is submitted that better results would be obtained if the specifications for steel could be made absolutely identical with those recommended by the American Institute for Steel Construction, which are also in line with the recommendations of the Society's Special Committee on Stresses in Structural Steel.

LEWIS D. RIGHTS,‖ M. AM. SOC. C. E.—The speaker will confine his remarks to stresses in structural steel. The Majority and Minority Reports of the Society's Special Committee on Stresses in Structural Steel¶ and the specification of the American Institute of Steel Construction have been mentioned, and Dean Turneaure has spoken of the Tentative Report of the Building Code Committee of the U. S. Department of Commerce.

The Tentative Report of the Building Code Committee issued in May, 1926, called for a basic unit working stress in tension of 16 000 lb. per sq. in.,

* Discussion on the Symposium on Unit Stresses in Structural Materials continued from October, 1926, *Proceedings*.

† Chf. Engr., McClintic-Marshall Co., Pittsburgh, Pa.

‡ *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.

§ *Loc. cit.*, September, 1926, Papers and Discussions, p. 1424.

‖ Vice-Pres. and Contr. Mgr., Shoemaker Bridge Co., New York, N. Y.

¶ *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 392.

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The Tentative Report of the Building Code Committee issued in May, 1926, called for a basic unit working stress in tension of 16 000 lb. per sq. in.,

* Discussion on the Symposium on Unit Stresses in Structural Materials continued from October, 1926, *Proceedings*.

† Chf. Engr., McClintic-Marshall Co., Pittsburgh, Pa.

‡ *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.

§ *Loc. cit.*, September, 1926, Papers and Discussions, p. 1424.

‖ Vice-Pres. and Contr. Mgr., Shoemaker Bridge Co., New York, N. Y.

¶ *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 392.

whereas the revised Tentative Report issued in September, 1926, raised this tension working stress to 18 000 lb. per sq. in.

The speaker takes this occasion to congratulate the Building Code Committee on this change of view as a step in the right direction. He feels that the Committee might have gone still further and raised this tension working stress to 20 000 lb. per sq. in. which is the value recommended by the Majority Report of the Society's Special Committee on Stresses in Structural Steel.

It may be well to say a word regarding this question of a change of view, which after all seems to be a matter of education. For instance, the Special Committee on Steel Columns and Struts of the Society, of which the speaker was a member and Chairman, made a report in 1918, recommending a maximum column working stress of 12 000 lb. per sq. in.* This report was tentative because the work had to be closed up somewhat hastily due to the demands of the World War. A little later a Joint Conference Committee of the Society and the American Railway Engineering Association, of which Dean Turneaure was Chairman, agreed on 12 500 lb. per sq. in. as the maximum working stress for columns.

At that time engineers were somewhat influenced and perhaps disturbed by results of tests on thick materials, and it was thought well to lean on the conservative side. Since then they have gained more confidence and Dean Turneaure has stated that he would favor a maximum working stress for columns 1 000 lb. higher than that recommended by the U. S. Building Code Committee, or 15 000 lb. per sq. in.

The majority of the Society's Committee on Stresses in Structural Steel has gone still further and has recommended a maximum working stress for columns of 16 000 lb. per sq. in., to correspond with a tension stress of 20 000 lb. per sq. in. The speaker is glad to endorse this change to higher stresses as a step in the right direction, which is warranted by present knowledge.

RUDOLPH P. MILLER,† M. AM. SOC. C. E.—The Building Code Committee of the U. S. Department of Commerce, in its latest revision of the prospective report on working stresses, has, it is believed, taken into account most of the criticisms brought out in this Symposium. If some of that Committee's conclusions do not coincide altogether with those of the Engineering Profession, it must be remembered that conditions as they exist throughout the country must be taken into account. Official supervision varies considerably in quality and adequacy. In some cases, it is perhaps not at all to be trusted. Conservatism in recommendations, therefore, seems to be justified.

It might be held that the logical treatment would be to permit the use of the highest working stresses in materials of construction in order to give the public the benefit of resultant economies, on the assumption that designers are competent and honest, but it is not necessarily the best method in the interest of public safety. The recommendations of the Committee are made with ample explanations to those who are formulating codes, and it is they who must judge of what is best for their own communities.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1583.

† Cons. Engr., New York, N. Y.

L. S. MOISSEIFF,* M. AM. SOC. C. E.—Probably the speaker is not exactly on the popular side of this question. He fears that there is danger of inflating unit stresses in America as well as dollars.

There seems to be such a prevailing optimism that it is natural to think everything is lovely. Bridge engineers, however, do not have to be reminded of the state of affairs that existed about two decades ago. At that time the younger bridge engineers at least thought it was absolutely impossible for any large structure to fail, and they had a bitter lesson.

The speaker agrees, without any question, that design should be improved, that loads should be differentiated, and that secondary stresses and details should be well worked out. All engineers believe that and know that it is the method of scientific design. On the other hand, however, they should not build up an engineering Utopia.

For example, a theoretical case has been built up on the past behavior of steel, on the manufacture of a good grade of steel, and on a pre-supposition that good design will be used, and excellent inspection. In all steel-making that is easily possible, but not all steel construction in buildings will be competent. If it were, unit stresses should go up, and no objection could be raised. To assure this still more information on the behavior of steel should be made available, such as the behavior at the yield point, or, better, the proportional limit. It is necessary to make a study of the actual development of details and erection as carried through in the larger cities.

People who observe large buildings being erected will conclude that engineers cannot well assume the perfect conditions that the case pre-supposes. It is all very well to write and state that it is based on good design, perfect material, fine inspection, and on the integrity of every man.

There is said to be a sure way of treating a mad dog. A certain Latin formula will stop him immediately when he hears it—the only trouble is that the dog does not understand Latin. Now it may be that the details may not understand the designs. To speak practically, what are engineers really after? They do not want to build structures with more material than is considered necessary, and they want to give the public the benefit of the saving that may be obtained by raising the allowable stresses for buildings.

Incidentally, the steel manufacturers thereby might be enabled to compete successfully with the builders of concrete structures. The latter, in their turn, would try to raise their unit stresses so as to exceed steel in economy. Considered from the competitive point of view the entire attempt is founded on shifting sand, because the cost of production of steel and concrete buildings is dependent on various labor and wage conditions. For instance, an increase of $1.00 per day for the well-organized concrete union men will suffice to unbalance completely any advantage obtained. On the other hand, a raise in wages of the workers in the steel mills might swing the pendulum in the other direction.

How much economy will be effected by the projected increase in unit stresses for steel buildings? The speaker has made approximate estimates

* Cons. Engr., New York, N. Y.

based on the cost of high buildings erected in New York City, and finds that an increase of 10% in the unit stresses of steel will result in a saving of about 1 to 1½% of the total cost of the building. Surely even such an amount when cumulative is worth saving; but from the point of view of the realty speculator who is erecting the building, it becomes a matter of secondary importance. It will not be given more attention than a variation in the necessary trim, the tiling of the toilet rooms, or the most convenient flooring. Why, then, attack the very life of the structure, its supporting skeleton, by trying to make savings that are comparatively unimportant? With more knowledge, with better control of materials, with a much better organized inspection system, these savings will be effected in due time.

It has been pointed out that Germany has raised the unit stress from its traditional 14 000 to 17 000 lb. per sq. in., or more. Americans should be happy that they do not need to do that. The United States is the richest country in the world, and there is no reason why Americans should experience the economic strains that Germans are compelled to undergo. They do it, because they have to do it. The entire work of standardization which is being done in Germany and Russia is the result of their poverty. It is true that Americans should not waste, but they should not save where safety may be concerned.

They should build well and permanently as befits their economic standard, and engineers should show their good judgment in a better way than by trying to save 1% of the cost of a structure at the expense of its strength. These thoughts have been expressed to help illuminate the problem from another point of view and to attempt at least to moderate the strong tendency to increase the present unit stresses.

Gustav Lindenthal,* M. Am. Soc. C. E. (by letter).†—The writer has held for a long time to the principle that a single unit stress should be used as a basis for dimensioning all members in a steel bridge, rather than a multiplicity of various units for various members. He also holds that the cross-section of each member should be determined for any possible combination of maximum stresses from dead load, live load, impact, wind, braking forces, centrifugal forces, and from bending and secondary stresses, the total maximum being divided by the basic unit stress to give the net cross-section of the member. The basic unit stress is taken at not less than 33% of the average ultimate strength of the steel. The method may be illustrated by citing the rules for dimensioning two heavy railroad bridges, the Hell Gate and Sciotoville Bridges, as follows:

Every bridge member shall be proportioned for that combination of maximum stresses that may occur in it simultaneously.

$$\text{Area} = \frac{D + L + I + \text{Lat.} + \text{Exc.}}{u}$$

in which,

D = dead load.

L = live load.

* Chf. Engr., North River Bridge Co., New York, N. Y.

† Received by the Secretary, September 28, 1926.

$$I = \text{impact} = \frac{L^2}{D + L} \times \frac{1\,200 + \frac{a}{n}}{600 + 4\,a}$$ (a = length of train behind locomotive tender for position of maximum stress, in feet; n = number of tracks loaded for maximum stress.)

Lat. = lateral forces.

Exc. = excess = (W + Br. + Temp.) − 20% (D + L + I + Lat.).

W = wind stresses.

Br. = braking stresses.

Temp. = temperature stresses.

u = unit stress.

The unit stress, u, for tension in the Hell Gate Bridge was 24 000 lb. per sq. in. for steel of 71 000 lb. average ultimate strength, and 20 000 lb. per sq. in. for steel averaging 66 000 lb. ultimate strength.

The writer lays particular stress on the rule for impact, which is the stress about which engineers actually know the least. No steel bridge can wear out under a quiescent load; but impact can break down any structure, if it is not made strong enough to resist. It causes dynamic stresses, the intensity of which cannot be expressed in static terms, which are the only terms used in bridge calculations. The writer's impact formula is empirical, but scientifically derived* as near as may be from the well-known experience with the effect of shock on car and locomotive axles and rails. When the fiber stress (as calculated from the bending formula) is less than one-fourth the ultimate strength, these parts do not last long. If the fiber stress is decreased to one-eighth they will last longer; and if from one-ninth to one-tenth of the ultimate, they will not break at all, except through a flaw in the steel. Such behavior of steel under shock cannot be deduced by any *a priori* methods of analysis, but can be determined only from experience.

Any shock in the axle and in the rail from whatever defect in the track, is communicated, of course, to the bridge which carries the load. The impact formula is designed to follow that shock to each member and to proportion its cross-section accordingly. It applies rationally to any length of span from 3 to 3 000 ft., or more. The whole calculation is simplified on a basis as near to actual conditions as can practically be defined. The usual formula for impact in railroad bridges provides, in the writer's judgment and observation, not enough steel in the floor construction and too much in the chords; that is, a certain amount of steel is in the wrong place. Long-span railroad bridges proportioned to the usual specifications require, therefore, more steel than need be. It is good practice to proportion such important bridges for the probable maximum locomotive and car loads—using, however, for such check calculations a fiber stress of 45% of the ultimate strength.

Robert C. Strachan,† M. Am. Soc. C. E. (by letter).‡—The Final Report of the Special Committee on Stresses in Structural Steel§ has brought forcibly

* *Engineering News*, August 1, 1912.

† Brooklyn, N. Y.

‡ Received by the Secretary, October 6, 1926.

§ *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 392.

to the attention of the profession the matter of revising, with due regard to known facts, currently accepted practice regarding allowable unit stresses. The data cited in the report show clearly that in presuming on a high degree of uniformity in physical qualities of the material as rolled to specifications, and an average strength warranting higher unit stresses than those heretofore accepted, engineers are fully justified.

The Committee is careful, however, to include in its recommendation, the condition "that the design is made by one competent to judge intelligently and correctly, the loads to be carried and the service to be performed * * *."

Engineering may be defined broadly as ingenuity of the highest type, directed in the light of scientific knowledge, toward producing or perfecting some tangible form of aid to man's happiness or convenience. Bearing in mind also that in practical affairs the consideration of first cost, upkeep, and renewal necessarily enters, it becomes evident that the condition laid down by the Committee is of the very essence of its report.

The practice of modifying loading requirements, assumptions concerning the mode of application of loads, specified effect of impact, and other factors proceeds from a proper conception of the engineer's true function. Through these modifications, unit stresses differing from the "basic" may in effect be used, and such practice constitutes an important means of applying the lessons of experience.

It is not to be expected that the adoption of the recommended increases in working stresses will lead to a proportionate decrease in the weight of steel structures; for, in specifying loads, the designer will continue to use his expert knowledge of basic requirements, to the end that his critical judgment shall be satisfied with the completed construction, and that the use of "excessive or misapplied mathematics" characterized by George F. Swain, Past-President, Am. Soc. C. E., as the worst engineering mistake, may not be charged against his methods.

To sum up, the net effect of the general acceptance of the Committee's recommendations should be to make possible a better balance of design through the extension of the field governed by the engineer's judgment.

Charles S. Whitney,* M. Am. Soc. C. E. (by letter).†—The writer believes that in determining safe unit stresses for structural materials the matter of building code enforcement and competency of designers should be left out of consideration. The ignorance of unskilled designers should not be allowed to prevent the efficient use of materials by competent designers. The custom of specifying low unit stresses in building codes, because incompetent designers are allowed to practice, encourages poor design and is dangerous. The unit stresses specified in municipal building codes cover a wide range, but the writer believes that there is no relation between the number of failures in the different cities and the intensities of stress allowed in their codes. Collapses are due to gross errors in design or construction.

There is another common type of failure in service resulting in cracking and disintegrating of the structure, which may be caused by the use of

* Cons. Engr., Milwaukee, Wis.

† Received by the Secretary, October 15, 1926.

unsuitably high unit stresses. Any one who has designed a large variety of structures will know that the proper intensity of working stress varies considerably with the character of the structure and the service for which it is designed. This fact makes any general specification of allowable working stress unsatisfactory in its application to many particular cases.

The recently recommended increase in working stress for steel structures is logical and proper but most emphatically the same reasoning cannot be applied to both steel and reinforced concrete structures. In the first case, the steel alone is carrying the load and any cracking of the coating due to deformation of the steelwork is superficial and does not endanger the safety of the structure. It is, of course, important in a building that the proportions of the members be such that deformations will not produce unsightly cracks in masonry and plaster, but that requirement need not prevent the use of higher unit stresses in structural steel.

In a reinforced concrete structure, high unit stresses in the steel will cause cracking of the concrete which seriously endangers the safety and integrity of the structure. These cracks must be accompanied by the slipping of reinforcing bars permitting the entrance of moisture and the disintegration of the concrete and steel. This action is augmented by the shrinkage which occurs in all concrete after setting, increasing the cracking and deflection of concrete beams. The writer has noted several cases where the shrinkage of concrete in very shallow beams, otherwise properly designed, has caused continued deflection and cracking of masonry walls or plastered partitions through a period of two or three years after the completion of the building.

Arguments have been put forward that the actual stress in steel reinforcement is less than the computed stress and that many structures in satisfactory service were designed with high unit stresses. The effect of the tension in the concrete in reducing the steel stress decreases as the ultimate strength is approached and it therefore does not materially increase the margin of safety. It does help to reduce the cracking of the concrete under working loads.

It has been found that visible cracks occur in concrete beams when the computed stress reaches 7 000 or 8 000 lb. per sq. in. The writer has in mind two reinforced concrete structures in which beams were badly cracked. He found that these beams were not properly designed for the loading specified in the building code, but that they had never been submitted to any load which would produce a computed stress in the steel greater than 18 000 to 21 000 lb. per sq. in. Most of the load in these cases was permanent dead weight. It would appear, therefore, that those concrete structures which are designed for high stresses actually may not have been subjected to the design loads or that they have been covered so that the cracking is not apparent.

The allowable stresses for concrete structures should be determined with these considerations in mind. The cracking of concrete under its design load should not be excessive. Cracking is more serious in exposed structures than in enclosed buildings. It would also seem advisable to use a lower stress when the proportion of permanent load is high than when it is low. With a high permanent stress in concrete beams, the plastic flow of the concrete, together with the shrinkage in seasoning and the extension of the steel, may cause excessive deflection and cracking.

WATER-PROOF MASONRY DAMS

Discussion*

By Messrs. Karl R. Kennison, P. A. Beatty, R. W. Gausmann, and H. F. Dunham.

Karl R. Kennison,† M. Am. Soc. C. E. (by letter).‡—Although the author has called attention to the necessity of considering uplift in the bed-rock itself as well as in the masonry, this necessity is not sufficiently emphasized. The importance of this point should be stressed because a tendency has often been noted in such designs to assume that uplift could be entirely eliminated by proper preparation of the foundation or a proper contact between the masonry and bed-rock.

The author apparently assumes that the foundation will be tight; "else why build on such a foundation?" The usual problem, however, is to take the best foundation available, usually without much choice, and build the dam to suit the conditions.

In analyzing the stresses in a dam the writer has never emphasized the distinction between the masonry and the bed-rock, but has always seen to it that the lines of pressure could be safely carried well into the bed-rock. Having this in mind, the proper distribution of dead weight is required above any plane in the bed-rock, as well as above any similar plane through the masonry, and on that account it would be difficult to save as much yardage of masonry as the author indicates because the weight is needed on the bed-rock.

The writer does not recall any failure of a concrete gravity dam by overturning on a plane well up within the masonry itself. On the other hand, such dams overturn on their foundations or even on a plane below their foundations, the artificial concrete being more suitable, as far as preventing wholesale uplift and overturn is concerned, than the bed-rock itself, which is seldom (if ever) a dense homogeneous mass.

However, even if the cost of the up-stream water-proofing proposed cannot be saved in reduced yardage of masonry, it is, of course, very desirable to secure this water-tightness at the up-stream face, particularly to prevent slight seepages which can prove very unsightly without endangering stability and which in freezing weather can build up the full reservoir head and break off parts of the down-stream face. The cost of such water-proofing should

* This discussion (of the paper by W. Watters Pagon, M. Am. Soc. C. E., published in October, 1926, *Proceedings*, and presented at the meeting of November 3, 1926), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Designing Engr., Metropolitan Dist. Water Supply Comm., Boston, Mass.

‡ Received by the Secretary, October 9, 1926.

be compared in each individual case with the cost of collecting seepage within the dam by various familiar methods.

P. A. Beatty,* M. Am. Soc. C. E. (by letter).†—In the Loch Raven Dam of the Baltimore, Md., water supply, on the Gunpowder River, designed, in part, and constructed under the supervision of the writer, a drainage system was installed to eliminate uplift.

This dam had a spillway height of 100 ft. above rock, and was constructed in 50-ft. sections, thus giving expansion joints at 50-ft. intervals along the axis of the dam. A carefully constructed facing of 1:2:4 concrete, erected in 8-ft. vertical lifts, extended from the bottom of the cut-off trench to the spillway crest, the sections being keyed at the expansion joints by two deep keyways with asphalt painted faces, and thoroughly bonded with the backing, which consisted of 1:3:6 concrete, containing about 25% of pudding stone in sizes up to 1 cu. yd., and also constructed in 8-ft. lifts. On the line of the expansion joints, at the junction of the facing and backing, vertical shafts, 30 in. square, extending from the first horizontal joint above the rock line to 8 ft. below the crest, were drained to the down-stream face of the dam.

From shaft to shaft was cast a groove, 14 in. wide and 7 in. deep, at each horizontal construction joint, and in this were laid porous concrete blocks, 3 ft. long and 14 in. square, perforated from end to end with a 6-in. circular opening, forming a free passageway between the shafts, the theory being that water seeping along the construction joints would find and follow this line of least resistance to the shafts, that coming through the expansion joints following directly to the shaft, thus making impossible the building up of pressure within the structure of the dam. Drainage was provided also at the rock line to intercept any flow passing beneath the cut-off wall.

The writer has no knowledge of the use of a water-proof metallic membrane in connection with any but rock-fill dams, but the saving in concrete pointed out by Mr. Pagon will undoubtedly direct further attention to this mode of construction. Practical consideration of the matter suggests the following questions to be studied:

What danger might result from the difference in the coefficient of expansion between the concrete and the metal used?

What would be the result of very small leaks in this membrane considering the numerous ties required to bond the two sections, that is, the backing and the protective coating, and the danger of perforation during construction?

In case the pool dropped, exposing a considerable area of the dam during very cold weather, what effects might be expected from successive freezings, recognizing the tendency of moisture to retire before cold, through a porous material, until an impervious surface is reached?

R. W. Gausmann,‡ M. Am. Soc. C. E. (by letter).§—The author presents an interesting solution of the problem of gravity dam design. It is gen-

* Cons. Engr., Camp Hill, Pa.

† Received by the Secretary, October 29, 1926.

‡ Hydr. Engr., Ulen & Co., Athens, Greece.

§ Received by the Secretary, November 4, 1926.

erally accepted that if water can be prevented from entering the up-stream face of a gravity dam no provision need be made for uplift pressure. The design proposed would undoubtedly accomplish this result providing a membrane could be made and kept absolutely water-tight at all times. It would be rather difficult to fabricate and test this membrane under field conditions and also protect it during the pouring of the concrete.

In addition, ample provision would have to be made not only for the expansion and contraction of the masonry but also for the gradual settlement that would occur during construction. Granted that all this might be accomplished, it is still a question whether the saving is as great as has been estimated by the author.

Generally speaking, masonry dams are not the same height throughout their length, in fact, the maximum height extends only for a short part of the length. This is particularly true of high masonry dams, therefore, the assumed saving should not be computed by multiplying the maximum section by the length as has been done by the author, but should be determined for each individual case.

A typical dam was computed using the author's graphs and costs and an actual profile across a valley. In this computation no allowance was made for foundations. The dam, constructed with a membrane, showed a saving of 6.0%, whereas the author's method shows an apparent saving of 12 per cent. A rough computation of the probable cost of a lead membrane (considered to be the most satisfactory) reduced the saving from 6.0% to a little more than 2 per cent.

It seems to the writer that the masonry outside the membrane should not be considered a part of the structure proper unless a much greater number of ties was used than is indicated on Fig. 3*; it should rather be considered as a veneer or protection for the membrane. If this is the case then the dam must be widened by the thickness of this protective masonry and this added width will reduce the saving to almost nothing.

Should the membrane be punctured by some unforeseen contingency or if, due to faulty workmanship or poor material, it should not be absolutely water-tight, there is no provision in the design to care for the uplift which is bound to occur.

H. F. DUNHAM,† M. AM. SOC. C. E. (by letter).‡—Of the three features "not to be overlooked" it appears that uplift should be regarded as the least troublesome or uncertain. At Berea, Ohio, there is a deep sandstone quarry. In the fall massive blocks, 2 cu. yd. or more in size, may be taken from the bottom of the quarry, shipped hundreds of miles, and properly set as ashlar masonry. Then after the first freeze each block will be found neatly divided on a horizontal plane. This fact is mentioned to show that under pressure at great depths and in long periods of geologic time the sandstone becomes saturated with water much more thoroughly than it would ever be near the

* *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1576.

† Civ. and Hydr. Engr., New York, N. Y.

‡ Received by the Secretary, November 8, 1926.

earth's surface. The quarrymen know that the output from the bottom of the pit should be "seasoned" before it is sold.

Now imagine a dam of any reasonable cross-section carved out of like saturated sandstone, free from cracks or seams but subject to water pressure throughout. The conditions for promoting uplift would thus be ideal or perfect. Pass a horizontal plane through such a dam at any elevation and the uplift necessary for a change in position would be measured approximately by the weight of that part of the dam above the plane plus the cohesive strength of the sandstone. The resistance to uplift would be definite and well co-ordinated, like the weight and wind stresses in an anchorage. Substitute concrete, or cyclopean or other masonry, for the natural rock and although saturation might not be so perfect the tensile strength of the material as a mass would become an essential if not a controlling factor.

The point is that uplift is a variable. Bad construction, great uplift; good construction, small uplift! This applies to foundations and anchorage as well. The Austin (Tex.) Dam gave way after a 5-in. rainfall and when the water was higher than previously known. At that time the vertical difference in elevation between the water below and above the dam was doubtless less than it had been before. That vertical distance was the measure of the uplift due to pressure and was also less than before. The high water lessened the weight of the dam and its force moved a part of the structure down stream.

Back-water at a flood stage of any river, even the Niagara, is noticeable. In a Kansas river a dam 12 ft. high could not be located by the appearance of the surface of the water directly above it during a flood stage. The difference between low and high water was about 32 ft. Under those conditions there was no uplift.

THE DESIGN, CONSTRUCTION, AND OPERATION OF A SMALL SEWAGE DISPOSAL PLANT

Discussion*

By H. L. Thackwell, Assoc. M. Am. Soc. C. E.

H. L. Thackwell,† Assoc. M. Am. Soc. C. E. (by letter).‡—This plant is typical of many similar layouts in Texas, with the exception that the topography of Texas is generally such that little head is available for a gravity plant. Consequently, sewage disposal plants along the bottom-lands, at the confluence of streams, are usually protected by levees; and gravity drainage during high water is impossible.

Imhoff tanks, usually subject to foaming, should be arranged with wash-watered troughs situated within the gas-vent area, so that by raising the sewage level the scum can be washed into the trough and out to the sludge drying beds. This has been accomplished in a crude way in Texas by forcing a rubber plunger below the Y-or T-connection in the sludge pipe and cutting off the riser sludge pipe below the surface of the sewage. When the sludge outlet valve is opened, the surface scum can be washed into the riser pipe and out into the sludge discharge pipe. Chester Cohen, Assistant Sanitary Engineer of the State Department of Texas, has proved that the use of chlorine in the influent of the Imhoff tank at Lufkin, designed by the writer, is a specific for foaming.

The septum walls of the Imhoff tank should be reinforced to withstand an unbalanced pressure, a hydrostatic head of at least 4 ft. The tension side of the slab would be on the side of sedimentation. There is a relationship between sludge area, transition area, and gas-vent area. These areas, if proportioned correctly, will control the shape of the outside walls. Fine screens placed in front of the effluent pipe are frequently required where baffles or primary screening are not effective in retaining chewing gum, watermelon seeds, matches, grease balls, etc., which tend to plug the trickling filter nozzles. At Electra, Tex., the operator spends a considerable part of each day taking balls of chewing gum out of the sprinkler nozzle. The chlorination of sewage flowing through the tank, if of sufficient dosage, has a salutary effect on the odors arising from the sprinkling nozzles.

In regard to the trickling filter and dosing chamber, it would seem that much better distribution of flow could have been obtained by building a central

* This discussion (of the paper by Franklin Hudson, Jr., Jun. Am. Soc. C. E., published in October, 1926, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. and Const. Engr., Rusk, Tex.

‡ Received by the Secretary, October 13, 1926.

dosing chamber supported on columns and having a manifold main running in two directions. The influent to the dosing chamber could have been supported on a trestle. The losses of head then would have been more perfectly distributed and the dosing tank could have been hopper-shaped without interfering with the spray from the nozzles. The floor of the tank, if of ridge and valley construction, with a difference of about 10 ft. between ridge and valley (the valley being covered with a perforated split tile), would take care of the under-drainage more economically than tiles covering the entire floor area.

The secondary sludge settling basin, or humus tank, is coming into more general use in connection with trickling filters. The sludge from this tank, however, unless absolutely stable, should be returned to the primary digestion tank rather than directly to the sludge beds. It is to be regretted that the author failed to include any of the standard tests familiar to all sanitary engineers connected with such work. Without such tests, nothing can be said as to the efficiency of the various units constructed, or as to the general plant efficiency for reduction of suspended matter and bacteria. The idea of having an incinerator adjoining the screen chamber is a good one, and is used in only a few plants in Texas. The author is to be commended in that his plant seems to be of ample size, and should do very well for some years to come for a population of 3 000.

The writer's experience as to the efficiency of trickling filters when capitalized against their cost, is that they are not worth the money. Among the several outstanding features against the use of trickling filters may be mentioned:

1.—The large area required for filtration space;

2.—The high initial cost;

3.—The odors emanating from the sewage sprays;

4.—The flora and fauna having the trickling filter as their habitat; and

5.—The periodical sloughing of partly oxidized matter that has to be taken care of, either in humus tanks or by discharging directly into the stream.

If the interest on the investment in a trickling filter were used in analyzing the operating cost, in comparison with that of the bio-æration type, the balance would be in favor of bio-æration, even when power charges are as high as 2½ cents per kw-hr. Apparently, engineers are too ready to recommend this type of plant to small municipalities that do not otherwise employ a regular operator. It is far better policy to use a certain amount of machinery or mechanical equipment that requires the attendance of an operator at least a part of each day, than to build a type of plant that (the municipality may think) can take care of itself.

The writer has recently designed and constructed a plant of 65 000 gal. capacity, providing (1) a grit chamber and coarse racks, with an adjoining incinerator for screenings; (2) a primary sedimentation tank with a removable Monel metal coarse screen; (3) a secondary sedimentation tank with an adjustable Monel metal fine screen; (4) sub-irrigation for the effluent with a

device for raising or lowering the underground water-table; and (5) final aeration and dilution in the stream with provision for chlorination if necessary. The sludge from the primary and secondary tanks, which is to be settled with lime as a coagulant, will be pumped into a separate sludge digestion chamber having a gas-proof cover and insulated side-walls and dome, from which the gas is drawn off and burned under a water-heater. The hot water is then circulated by means of copper tubing back through the interior of the sludge chamber, which treatment, in turn, causes more gas to be formed by raising its temperature. The cycle is thus repeated. The digester and heater are regulated by an automatic thermostat and temperature recording devices and are equipped with gas traps, pressure regulator devices, an air inlet, and blow-off valves.

The digested sludge is dried on a sand bed with a standard "glass-over" construction, the dried sludge being removed in dump cars on an industrial track. The primary tank, pump and heater chamber, and digestion tank are housed in brick buildings. The secondary tank is surmounted by a concrete pergola, supporting brass pulleys which are used in hoisting the screens. The tank is bordered with ornamental tile walks and has the appearance of a formal sunken garden, since it is lower than the surrounding grass-covered berms. The whole plant is parked with shrubbery and evergreen vines and is protected with a Page non-climbable fence. Water at a pressure of 60 lb. per sq. in. is provided for the use of the operator in cleaning the tanks and irrigating the lawn.

The cost of this plant, including an entrance roadway, 900 ft. long, with bridges and culverts, a water pipe line, 1 200 ft. long, and the engineering and constructing fee, was $10 000. Operation has not yet been started (October, 1926), since to date only a few connections to the sewer system have been made. This odorless type of plant was deemed necessary since the site is close to the local college grounds, the boys' dormitory being only 900 ft. distant.

TOWN PLANNING AND ITS RELATIONS TO THE PROFESSIONS INVOLVED

Discussion*

By Thomas Adams, Esq.

Thomas Adams,† Esq.—It would have been interesting to have heard more of Mr. Nolen's reasons for difficulties in regard to density and garages. It is the discussion of the difficulties found in special cases of this kind that gives a conception of the methods for treating these problems under ordinary conditions. Similarly, useful would have been some indication of rents or costs in relation to turnover, because the value of Mariemont as an object lesson of development, is in proportion as it demonstrates how it can be carried out elsewhere.

The city planner and the engineer have an obligation to society to show that whereas what they carry out is more beautiful and more idealistic, it is also, in its application elsewhere, more profitable from a financial point of view.

There is a query on one point mentioned by Mr. Nolen. What is wrong with the modern city? It is not that it is too big but that it is wrongly planned and wrongly developed. It is no worse to live in the crowded districts of Manhattan or of London with their congested streets than in the crowded tenements in a small city or village. What is wrong is not that cities are too big but that there is an improper balance in their growth. If the Garden City or Mariemont could be repeated a thousand times where there was ample space and ample opportunity for light and air, there would be no need to worry about the size of the area that would be covered by a group of such communities.

The speaker envies Mr. Nolen and his associates in the opportunity presented by this plan; the more so because one or two of his failures have been exactly in cases of this kind. No effort was spared to make a plan that seemed satisfactory and that would be the occasion of considerable satisfaction; and yet, owing to the wrong architect being employed or to some form of bad management, it was anything but a success. The necessity for collaboration between the landscape architect, the architect, and the engineer is demonstrated in such instances.

Sometimes the controlling management, in the interests of false economy, has destroyed the opportunity to make a success. For instance, the busi-

* Discussion of the paper by John Nolen, Esq., continued from November, 1926, *Proceedings*.

† Gen. Director of Plans and Surveys, Regional Plan of New York and Its Environs, New York, N. Y.

ness administration should not be under the city planner, but neither should the city planner be under the business administration. They should be two separate organizations, dovetailing together, working together, with possibly a final arbitrator in case of differences; but there should be no subordination of the technical expert in matters of design.

As to the plan—no one is in a position to make intelligent criticisms of a plan unless he has gone through the same experience and obtained the same knowledge of the site as the man who has made the plan. There can be only congratulations for Mr. Nolen, not only on the opportunity he has had at Mariemont, but on the apparent technical perfection of his work.

All agree as to the desirability for more collaboration between the landscape architect, the engineer, and the architect. How can this be brought about? The meeting at which this paper was presented, was held under the auspices of the American City Planning Institute in collaboration with the City Planning Division of the Society. There also exists a group of the American Institute of Architects who are interested in something that is different in name but really is the same thing—"community planning." Another effort should be made to bring about the needed collaboration. That may be accomplished somewhat in the way it has been done in England in framing the Constitution of the British Town Planning Institute. The basis of that Constitution is this: A person may be a full member of the British Town Planning Institute if he is a full member of the Institution of Architects, the Institution of Civil Engineers, or the Surveyor's Institution; or he may be a legal member if he is a qualified lawyer. However, he must have a degree in one of these four standard professions before he can be a member of the City Planning Institute. This is the general rule—but provision is made for election of distinguished men who have special qualifications, other than membership of the four basic Institutes or Societies.

The American City Planning Institute should be re-organized and developed on a strictly professional basis, with due regard to bringing in the architect, the engineer, the landscape architect, the lawyer, and, on a few exceptional occasions, the sociologist and others who are especially interested and who have already gained a certain standing. There are greater tasks awaiting the city planner in the future than in the past, but in order to deal with them he must be better equipped with technical knowledge and better organized.

GARBAGE DISPOSAL

A SYMPOSIUM

Discussion*

By Messrs. Harrison P. Eddy, L. L. Tribus, J. F. Jackson, and William A. Bassett.

Harrison P. Eddy,† M. Am. Soc. C. E.—The statement with reference to the general application of a single process to different conditions is very sound. It must be self-evident that the conditions in the different municipalities vary greatly. In his paper Mr. Greeley presented a summarized statement of the general conditions and methods of procedure, which establishes, perhaps for the first time, a basis or starting point for the consideration of garbage disposal that will be of great value to cities confronted by this problem.

One point in particular in his paper is of especial importance—the question of operation. As in many municipal undertakings, it is comparatively easy to secure the capital allowance for construction, but it is exceedingly difficult to obtain the funds for efficient operation. Included in that should be the matter of general upkeep and maintenance.

It is possible, with an inferior plant, to secure relatively good results if there is intelligent and efficient operation. Proper operation is probably the weakest point in the garbage and refuse disposal problem with which municipalities are confronted. The same problem is encountered in the matter of sewage disposal. Plants are built and then the funds for their proper maintenance and operation are rarely forthcoming.

One who is familiar with the plant at Rochester, N. Y., will recognize the soundness of that statement. After the plant had been in operation for relatively a few years (perhaps not more than five), that is, when Mr. Lewis first went there, it was almost on the point of being shut down because it could not be operated. That was largely due to lack of reasonable upkeep, of reasonable care, and of replacement of parts that were not functioning as they should.

Visiting that plant a year later one would hardly recognize it because of the repairs that had been made and the efficiency that had been obtained in operation. That accounts, in large measure, for the high maintenance cost in 1924 (about \$40 000 or \$60 000 for repairs), whereas during 1925 this had been reduced to about \$20 000.

* Discussion on the Symposium on Garbage Disposal continued from November, 1926, *Proceedings*.

† Cons. Engr. (Metcalf & Eddy), Boston, Mass.

The same is true with every method of disposal. Within two years complaints have been registered against a hog-feeding plant in Massachusetts. The State Department of Health had the matter under investigation. There were indications that the plant would have to be shut down, but by careful operation it has been continued and apparently it is operating successfully, as the complaints have ceased.

Similarly, with incineration, if the plant is not reasonably well operated, unsatisfactory results are certain to be obtained. It is possible that the matter of maintenance and operation is somewhat more difficult in a complicated and intricate plant, such as that described by Mr. Lewis, than in some of the others.

As a rule, incineration plants handle mixed refuse consisting of rubbish and garbage, rather than garbage alone. Misunderstanding of this fact might lead to a little confusion in a consideration of the several papers presented. Undoubtedly the most important feature of garbage disposal is the operation of the equipment provided.

L. L. TRIBUS,* M. AM. SOC. C. E.—There is a great deal in the Symposium that is worthy of discussion. One point made by Mr. Greeley in particular is a practical one that engineers who have such municipal problems in hand are constantly meeting, namely, as to how the average community selects a garbage destructor?

This is how it is usually done. An enterprising representative of some furnace will get the ear of an official and tell him that in such and such a place there is a very fine plant. The next step is a junketing expedition of inexperienced men who go to see it. The quality of the cigars is fine, the sandwiches, the salad—everything is excellent. The agent who puts up the best exhibit of this sort, wins for his company. After their virtual decision, the officials call in an engineer. If he is in accord with their determination he is just the finest fellow on earth, and the type of plant is adopted; if not, well the veil can be drawn. There is a great deal more of truth than fun in this method and practice: there is no need of enlarging upon it.

Mr. Eddy has spoken of the lack of care in the operation of equipment. All engineers have experienced such mismanagement. It makes little difference how well a sewage or garbage disposal plant is built, how "fool-proof", how perfect. After the first send-off and excitement are over, interest is gone. Operators will not give the plant the proper attention; it deteriorates, something happens. If there is intelligent official management, it is put in shape before it gets entirely out of control. If such management is lacking, it goes to pieces.

A great reduction plant was built on Staten Island, New York City—approximately 240 Cobwell tanks, splendidly constructed to handle 1 000 to 1 500 tons of garbage per day. Why did it not succeed? Although the speaker lives eight miles away, the stench was so great at times that it was necessary to close the windows.

* Cons. Engr. (Tribus & Massa), New York, N. Y.

He examined the plant many times, observing receptacles with broken inspecting glasses, so that, when the steam was turned on, the vapors naturally leaked out. When many such leaks existed, it made quite an accumulation of odors, and they could not be kept in the building.

Then, again, scow after scow—from 15 even to 20 in number—lay at the wharf for days with garbage, sometimes even a week old or more on arrival; rotting, filthy stuff, which, although it "smelled to heaven", the plant could not handle promptly.

The tankage building was not big enough to hold the cooked garbage, and, therefore, it was piled on the ground to take the rains and the sun, and give off generously of its caramel odors. That is why the great plant broke down—accumulated mismanagement, and planned with too much optimism as to capacity.

Extensive operating losses were due to the lack of ability to recover the solvents.

However, with proper construction, and painstaking management, such a plant might have succeeded, as that at Detroit, Mich., has apparently succeeded, and as that at Rochester is succeeding.

J. F. JACKSON,* M. AM. SOC. C. E.—These papers cover the field of garbage disposal from almost every angle, but some minor matters were not brought out which should receive consideration.

In Connecticut there are only two refuse disposal plants, one a reduction plant and one a destruction plant. In the majority of cases garbage is separated from refuse, most of the garbage is fed to the hogs, and the refuse is used for filling. One town uses a sanitary fill. The cost ranges from $0.14 to $1.56 per capita per year which shows the variation in the methods of collection and disposal. In many instances the hog farms are located in a different political subdivision from that in which the garbage is collected, and the work is done under contract. Where it is done by private contractors supervision ceases at the boundary line of the municipality. Few hog farms are conducted on a sanitary basis, and complaints are numerous and frequent. Consequently, the administration duties of the State Department of Health are increased and complicated.

It will be recalled that when control and regulation of the production of milk was started, it was confined at first to testing the milk as it was brought into the cities. Afterward, through the investigation of sources of communicable disease, this supervision was extended to the farms where the milk was produced, controlling the examination of the cows and the sanitary conditions in and around the stables.

Likewise, the supervision of municipal departments having in charge the collection and disposal of garbage could well be extended to include farms on which it is used. It is true that in some cases this is done, but only in a perfunctory manner. At a few municipal farms the arrangements for feeding the garbage to hogs are modern, and the sanitary conditions are satisfactory.

* Cons. Engr., New Haven, Conn.

At most farms lying outside the limits of the municipality they are quite otherwise and, at many, they are deplorable. In investigating slaughter-houses in Connecticut conditions were found that were positively shocking. This is a phase which, although minor, is very important as regards sanitation.

Another analogous feature in the disposal of garbage and the production of milk is the container. It is only in quite recent times that attention has been paid to the milk bottle, its cleaning and capping; but to-day this is quite an important sanitary feature of milk production.

Practically every one at one time or another has revolted at the disgusting condition of the ordinary garbage can. A few favored ones have escaped it by installing house incinerators and testify eloquently to this improvement. Some time must elapse before these will be made a part of the permanent fixtures of a house, but with the trend toward permanent installation of washing machines, refrigeration, etc., the incinerator's place in building construction can be seen in the not distant future. In the meantime much improvement can be made in the control of collection and disposal, and in the design and location of the receptacle.

WILLIAM A. BASSETT,* M. AM. SOC. C. E. (by letter).†—Previous to 1922, the Village of Scarsdale, N. Y., like many other residential suburban communities, was dependent on private contractors for the collection and disposal of its municipal wastes. This service at that time was handled by about a dozen individual contractors, some of whom restricted their collection to garbage alone, and the cost to the individual householder ranged from about $3 to $10 per month. Moreover, a considerable part of the population of that village was not furnished with this service so essential in communities changing from rural to urban characteristics.

Recognizing the need for unified service in this matter, under suitable control, a committee of the Village Board working with Mr. Boniface, the Village Engineer, made an extensive study of the problem of waste collection and disposal for residential communities, and as a result of this study, a collection force was organized, suitable equipment purchased, and the construction of a Beccarri garbage disposal plant was authorized. There were sound reasons for selecting this type of plant. Its simplicity, low cost of operation and maintenance, adaptability for extension through the construction of additional units, and apparent freedom from nuisance of any kind resulting from its operation seemed to indicate its peculiar suitability for a high-class residential district such as Scarsdale.

Conditions attending the operation of the plant have not fulfilled expectations in so far as freedom from nuisance, caused by objectionable odors, is concerned. As pointed out by Mr. Boniface, after the nitrifying cycle has been completed, and the material in the cells has been made ready for removal, the discharge of the cells is accompanied by highly obnoxious odors. Moreover, the material in the cell has a high moisture content, and is so compacted as to be difficult to handle. Apparently, the obnoxious odors are due

* Cons. Engr., New York Bureau of Municipal Research, New York, N. Y.

† Received by the Secretary, October 25, 1926.

to the retention of the gases incidental to fermentation by the moisture in the mass. After the material taken from the cell is dried in the open air, it is apparently stable and odorless. Unless these unsatisfactory conditions of operation can be corrected, there is grave doubt as to the applicability of the Beccarri system of disposal, at least in its present form, to meet American conditions.

The reasons for this are not hard to find. The quality and characteristics of garbage produced by communities in the United States differ materially from those found in Italian cities, in which the Beccari system is at present in successful operation. American garbage is high in grease, animal fats, and moisture; that produced in Italian cities is decidedly low in these qualities, particularly in its moisture content. This is due, in part, to the salvaging methods used in Italy in handling garbage before it is introduced into the cell. Moreover, it is stated that at the Italian plants the material when removed from the cell is dry, dark in color, and of about the consistency of humus or leaf-mould. It would not appear probable that the richness in fats and greases of the garbage produced by American cities would have any unfavorable influence in the operation of the nitrifying process. In fact, it is believed that the contrary might prove true. However, there appears to be little question but that the high moisture content of the Scarsdale product re-acts unfavorably on the process, and has been the main contributing factor in producing the nuisance conditions.

It is quite probable that some modifications in the present design of the Beccarri plant at Scarsdale should be made in the construction of other plants, but it is believed that the main feature required for successful operation of this process is to effect a substantial reduction in the moisture content of the garbage before the latter is placed in the cell.

THE NEW YORK STATE BARGE CANAL AND ITS OPERATION

Discussion*

By Messrs. Maurice W. Williams and T. Kennard Thomson.

Maurice W. Williams,† M. Am. Soc. C. E.—The increase in traffic on the Barge Canal since its opening is not generally understood. In 1918 the Canal was opened on its present line but not to full dimensions. It was not until 1919, therefore, that the Canal can properly be said to have been fully opened. In the first year of its existence, 1918, the tonnage was approximately 1 160 000; in 1925, it was 2 344 000, an increase of 100% in 7 years.

The average progressive rate of increase of traffic in each year over the preceding year since 1918 has been about 10.65%, and is at least equal to the average rate of growth of similar comparable waterways. The annual rate of growth of traffic on the waterways of the State was only about 7% in the early Thirties, when the total tonnage was about equal to what it is now on the Barge Canal. If the factor of 10% be applied progressively to the traffic of 1925 for estimating the tonnage of the present and succeeding years, it is found that the Canal will carry its nominal capacity of 20 000 000 tons per year in about 1948. The word "nominal" is used because nobody knows how much the Canal can carry; but the Erie, Oswego, and Cayuga-Seneca Divisions certainly can carry 20 000 000 tons per year, to say nothing of the virtually independent Champlain Division.

One of the main commodity items, which the Canal always has carried and probably always will carry, is grain. In 1925 grain formed about 40% of the total traffic. It is said in certain quarters that the exportable quantity of grain is bound to fall off so that in a few years there will be no grain to export. Such a conclusion is exceedingly questionable. The per capita production of wheat in the United States and Canada combined was about 7½ bushels per annum twenty-five years ago. To-day, it is about 10 bushels. In the last forty years the total production of wheat in the two countries has increased 50% faster than the population, and the wheat fields are still being pushed nearer the Arctic Circle. There is good reason for considering the grain of the United States and Canada jointly for purposes of export, in that the exportable grain of both countries moves through the ports of either with virtually equal facility. At present, there are exported annually from the ports between Montreal and Newport News, Va., inclusive, about 12 000 000 tons

* Discussion on the paper by Roy G. Finch, M. Am. Soc. C. E., continued from November, 1926, *Proceedings*.

† Cons. Engr., New York, N. Y.

of grain per year, most of which is susceptible of being handled by the Barge Canal. Of this amount at least 7 000 000 tons move in the seven full months of canal navigation, to say nothing of the days in excess of seven months when the Canal is open. There should be great quantities of grain to be moved for export as well as for domestic consumption for many years to come.

It has been brought out that the number of barges navigating the Canal has not been increasing. To-day there are slightly less than 800 barges. The number was about the same in 1924, but between 1922 and 1924 the average capacity of barge increased from 420 to 480 tons, or 15 per cent. During this period many old barges were scrapped and their places taken by new barges of much greater capacity. Sizes are still being increased, and some of the barges now almost fill the locks.

The speed of transportation by the Barge Canal compares favorably with that by rail. In October, 1925, the average car-mileage in this region was the maximum to date—slightly more than 33 per day. The report of the Superintendent of Public Works gives the mileage of the average barge navigating the Erie Division as 33.3 per day, virtually the same as by rail. The instance mentioned by Mr. Finch of the movement between Oswego and New York is in no way exceptional. Many other lines deliver nearly the same service and it may be expected that they always will.

There has been no decided movement of iron ore on the Barge Canal west from Lake Champlain. During 1925 and a year or two previously there was a movement of imported ore from New York to the Buffalo District and farther west; but it is too soon to make forecasts as to the development of this traffic. The movement of steel shapes and semi-manufactured iron from points near Buffalo eastward has developed in some quantity.

The average unit saving on freight actually transported on the Canal is estimated to be about $1.15 per ton. That figure is obtained by consideration of the known saving on grain and on certain other commodities, the testimony regarding which was clear to the Barge Canal Survey Commission. The saving is the difference between what it actually costs to ship by Canal and what it would have cost by the next least expensive route; the tonnage on which the saving is known is relatively large.

There are undoubtedly other advantages in the Barge Canal. It was also brought out in that testimony that the depression in freight rates in that section of the country due to the competitive influence of the Barge Canal effects an indirect saving to shippers amounting to $50 000 000, or more, each year. This figure is reached by a consideration of the tonnage produced and consumed in the area affected by the Canal and a comparison of the transportation rates near to and far from the Canal.

Undoubtedly the Barge Canal exercises a beneficent influence on the industry, commerce, and prosperity of the State of New York by assisting in maintaining the flow of commerce through the State. For example, it was brought out in the same testimony that if the Canal had not been available certain export grain would not have moved through New York State at all, but would have gone by some other route.

T. Kennard Thomson,* M. Am. Soc. C. E.—It is, of course, astounding to read that the best year's traffic on the New York State Barge Canal amounted to only one-third that carried on the old Erie Canal with its 7-ft. depth.

Two monumental blunders were made on the Erie Canal when it was recently reconstructed. The first was one of location in maintaining the hump of 57 ft. at Rome, which prevents any water from Lake Erie reaching the Hudson River. This requires the expensive storage basins at Delta and Hinckley to feed the Canal east of Rome, as well as to provide the additional lockage.

The second error was in making it a Barge Canal with fixed bridges having a minimum clearance of 15½ ft. This was a political as well as an economic blunder made in the vain attempt to get the Cities of New York and Buffalo to vote for the enlargement. To realize this, it is only necessary to inspect the Canadian canals and note the use there made of the unrestricted height of boats, permitted by movable bridges.

The short-sightedness of New York City and Buffalo in thinking that the loss of a comparatively few stevedores who now load or unload freight at those two places could in any way compare with the prosperity of the entire State resulting from the free use of the canal, is simply marvellous.

New York City pays 25% of the tax of the United States, which means that it owns 25% of the total wealth and is therefore vitally interested in the prosperity of every foot of New York State, as well as the remainder of the Continent (Canada and Mexico included).

For the same reason, New York City and Buffalo will vastly benefit when the (Barge) Canal is used to its limit and when the Niagara and St. Lawrence Rivers carry ocean steamers to the Great Lakes, utilizing at the same time the hydro-electric energy of these rivers. This energy will amount to 8 000 000 h. p., saving 80 000 000 tons of coal per year, or $640 000 000—about $2 000 000 per day—most of which is now going to waste. A further result will be the construction of twenty Pittsburghs on the Canadian border.

When the speaker was one of the Board of five Consulting Engineers for the New York State Barge Canal, he was often asked "whether the Barge Canal would ever pay." His reply was that if the State of New York were to construct skyscrapers in New York City for the public, without equipping them with elevators, lights, water, etc., the public would never use them, as long as up-to-date fully equipped buildings were constructed by private capital near-by. Active agents would see to it that prospective tenants of the State owned buildings were diverted to the privately owned structures.

For the same reason, when the State handles the Barge Canal as a business enterprise, going after the traffic, keeping the terminals, etc., up to date, the Canal will be of enormous value to the State and to other parts of the country.

* Cons. Engr., New York, N. Y.

When the speaker hears estimates of future traffic for 20 to 50 years hence, he is forcibly reminded of a diary written by his father in 1842, describing a trip from Buffalo to New York by stage coach, in which he stated:

"Shortly there will be a railroad all the way from New York to Buffalo * * *. It was interesting to hear the discussion of the fellow passengers. The majority said that there would never be traffic enough to keep the rust off that one pair of rails, while the more enlightened ones thought that the single-track railroad would carry all the traffic between those cities for one hundred years to come."

Almost any one predicting the street traffic of automobiles and other vehicles in New York City, 10 or 15 years ago, would have been nearly as greatly in error. Who would have believed, 20 or 30 years ago, that the New York elevated and subway lines would carry the number of people they now do; and, second, that the 5-cent fare for this enormous number of people would involve a loss. Traffic estimates based on horse cars would be useless for modern conditions, and yet estimates are frequently made for conditions that are similarly radically different.

The Empire State does not need to worry about the possible volume of traffic—freight or human. All that it need do is to provide the facilities, and see that it gets its share, by exercising ordinary business common sense.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

MORDECAI THOMAS ENDICOTT, Past-President, Am. Soc. C. E.*

DIED MARCH 5, 1926.

This memoir records the professional career of a Past-President of the Society, who was closely associated with Naval shore construction activities for more than forty years, and who was also an authority on Isthmian Canal projects.

Mordecai Thomas Endicott was born at Mays Landing, N. J., on November 26, 1844, the son of Thomas Doughty and Ann (Pennington) Endicott. He was a direct descendant of John Endicott, the first Governor, in 1629, of the Massachusetts Bay Colony. He received his primary education in the schools of the Presbyterian Church and through private instruction at Mays Landing, N. J. In 1864 he entered the Rensselaer Polytechnic Institute, at Troy, N. Y., where he pursued the regular course in Civil Engineering, and from which he was graduated in 1868 with the degree of Civil Engineer.

Following graduation he entered actively upon his engineering career. He was first employed as an Office Assistant by the late R. P. Rothwell, M. Am. Soc. C. E., a Civil and Mining Engineer, of Wilkes-Barre, Pa., from July until December, 1868. He next served as a Rodman in the Brooklyn Navy Yard until January, 1870, when he entered the service of the New Haven, Middletown and Willimantic Railroad Company as a Draftsman, with headquarters in New York, N. Y. After spending five months in the office he was detailed to the field as an Assistant on the construction of the foundations and approaches of the Connecticut River Bridge at Middletown, Conn. Mr. Endicott continued on this work until the latter part of 1870, when he accepted an appointment as Assistant Engineer in charge of the "Dresden Extension" of the Cincinnati and Muskingum Valley Railway, from Zanesville to Dresden, Ohio.

On the completion of this railroad extension, he was appointed, on February 1, 1872, in a civilian capacity as an Assistant Civil Engineer at the then newly established League Island Naval Station at Philadelphia, Pa., and continued thereafter in the Government employ. After rendering meritorious services for a period of eight months in the development of the new Naval Station and on the construction of public works and public utilities, he was given orders placing him in charge of the Civil Engineering Department of the Philadelphia Navy Yard, where he continued until he was commissioned a Civil Engineer in the Navy on July 13, 1874, and ordered to the Naval Station at New London, Conn.

* Memoir prepared by the following Committee: H. H. Rousseau, *Chairman*, and C. L. Strobel, Members, Am. Soc. C. E., and A. N. Talbot, Past-President, Am. Soc. C. E.

Following five years of service in charge of the Civil Engineering Department at the Naval Station at New London, where he designed and constructed wharves, buildings, and other improvements, Mr. Endicott was ordered, in 1879, to similar duty at the Navy Yard at Portsmouth, N. H. In 1881, he was sent to the League Island Navy Yard at Philadelphia, where he remained in charge of public works design and construction until 1886, when he was ordered to the Norfolk, Va., Navy Yard. Here, in addition to other work, he was in charge of the construction of the dry dock that was completed in 1889. While at Philadelphia, in 1882, he was given the rank of Commander.

In October, 1889, Commander Endicott was detached from duty at Norfolk, and appointed by the Navy Department as a member of a Board to consider and investigate conditions and necessities at the New York Navy Yard, and to report on a plan for its development and improvement. On completion of this Board duty in April, 1890, he was detailed as Consulting Engineer to the Bureau of Yards and Docks, United States Navy Department, in Washington, D. C., and placed in charge of all civil engineering work under the cognizance of the Bureau.

This duty opened up a unique opportunity for him, for, while the construction of modern naval vessels had begun, the Navy Yards had not yet emerged from the condition in which they were when wooden vessels were supreme. Carts drawn by oxen still met transportation needs. Electric installations for light and power had not been introduced. Street improvements were principally noticeable by their absence. Lumber was the material used for dry-dock construction, and brick and lumber for buildings. The day of steel and concrete for Navy Yard construction had not arrived. How well Commander Endicott availed himself of this opportunity to modernize the Navy Yards may be judged from the record of his subsequent career and advancement, during the sixteen years that passed while he was on active duty. During this period, 1890-1906, there was also a tremendous growth and advancement in all lines of engineering and industrial accomplishment and activity outside the Government service, and it was his constant aim that the Navy should keep up with the pace set by the country's industrial plants and establishments.

Commander Endicott, then 46 years of age, was eminently fitted for this assignment at the Navy Department, having had eighteen years' experience at the principal Navy Yards along the Atlantic Coast, during which period he had become thoroughly familiar with naval conditions and requirements. This detail Commander Endicott held until his appointment as Chief of the Bureau of Yards and Docks, with the rank of Commodore, on April 4, 1898.

During this eight years of service as Consulting Engineer in the Bureau of Yards and Docks one hundred and ten construction contracts were entered into, embracing miscellaneous and varied engineering structures. Among the outstanding projects may be mentioned a timber dry dock for the Puget Sound Navy Yard, the design for which included a concrete and stone entrance, a marked improvement over the standards previously followed; turn-tables for 40-ton cranes; timber dry docks at New York and Port Royal, S. C., with im-

provements similar to those adopted for the dry dock at Puget Sound; and the design of 40-ton locomotive jib-cranes capable of handling the armor plates required by the battleships then being constructed. On March 21, 1898, immediately prior to his appointment as Chief of Bureau, he was promoted to the rank of Captain, and by the provisions of the "Personnel" Act of Congress, effective March 3, 1899, he was elevated to the rank of Rear Admiral, which rank he held thereafter. He was the first Civil Engineer of the Navy to hold this rank.

Commander Endicott's reputation as an engineer, and his success in Naval civil engineering work led to his appointment by President Cleveland, on April 26, 1895, on the recommendation of the Secretary of the Navy, and without any application on his part, as the Navy member of the Nicaragua Canal Commission of 1895, headed by the late William Ludlow, Col., Corps of Engineers, U. S. Army, M. Am. Soc. C. E. The late Alfred Noble, Past-President, Am. Soc. C. E., was the third member. This Commission examined the route of the canal as proposed by the Maritime Canal Company, and reported on its feasibility, cost, and permanence. The Commission's report led Congress to delay taking action on bills furthering the Nicaragua Canal project which was eventually abandoned in favor of the Panama Canal route.

By Act of Congress approved July 19, 1897, the Navy was specifically forbidden by law to pay more than $300 per ton for armor for battleships; and if he could not purchase armor within this limit, the Secretary of the Navy was authorized to take steps to establish a Government factory of sufficient capacity to make such armor. The Act further directed the Secretary of the Navy to appoint an Armor Factory Board, to consist of competent naval officers, to advise and assist him in executing the authority conferred. Commander Endicott, then acting as Consulting Engineer to the Bureau of Yards and Docks, was selected as the Civil Engineer member of this Board. The report of the Board which went very fully into the design, construction, and the operation of a Government Armor Plant and which was accompanied by plans and estimates of cost, impressed Congress with the feasibility and economy of this project, and was largely responsible for the considerable reduction in the subsequent quotations for armor plate, as compared with the bids made prior thereto, as the armor-plate manufacturers then realized the Government's determination and ability to manufacture its own armor plate if they were unable to meet its views as to price.

In the spring of 1898, the four-year term of office of the Chief of Bureau of Yards and Docks expired, and a new Chief of Bureau was to be appointed. This country was then on the eve of the Spanish War, and the new incumbent would have to carry the work of this important Bureau through a war the magnitude and duration of which could not be predicted. The man most familiar with the work of the Bureau and the shore needs of the Naval Establishment was Commander Endicott, then on duty in the Bureau. To appoint him Chief of Bureau would necessitate breaking the traditions and precedents of the preceding fifty-six years during which time only officers of the line of the Navy had held this position. The precedent was broken. President

McKinley, on the recommendation of Secretary of the Navy John D. Long, submitted the name of Mordecai T. Endicott to the Senate for confirmation, and on April 4, 1808, he was appointed Chief of the Bureau of Yards and Docks, the first officer of the Corps of Civil Engineers to be honored with this appointment. The success of his first four years of administration of this office resulted in his re-appointment in 1902 by President Roosevelt for another four-year term, and he was given a second re-appointment by President Roosevelt in 1906.

He reached the then retiring age of sixty-two years on November 26, 1906, but continued at the request of the Secretary of the Navy to serve as Chief of Bureau until he resigned this office on January 5, 1907. He remained on duty in the Navy Department until March, 1907, when he was detailed to the Department of Justice as Technical Adviser to the Attorney General of the United States on suits being prosecuted against the Navy Department. On June 30, 1909, on the completion of this work, he was relieved of active duty and ordered to his home.

A few of the outstanding accomplishments of Admiral Endicott's nearly nine years of service as Chief of the Bureau of Yards and Docks should be mentioned. When he was appointed, the Corps of Civil Engineers comprised only thirteen officers. This small Corps was inadequate in numbers to meet the rapidly expanding requirements of the Navy. Through the discretionary power to make additional appointments vested in President McKinley, the Corps was increased to eighteen; and, later, on March 3, 1903, through the efforts of Admiral Endicott, Congress fixed the number of officers in the Corps of Civil Engineers at forty. The successful conduct of the office of Chief of Bureau by an officer of the Corps of Civil Engineers had demonstrated the wisdom of this policy.

It is natural to assume that a technically trained man with administrative ability is better qualified to handle the work of a strictly engineering bureau than one who has to rely on assistants for the technical advice and information on which he has to make his decisions. Admiral Endicott's efforts were untiring to promote the welfare of the Bureau and the Corps by securing the adoption of this practice as a permanent policy before he gave up the Bureau, and he was finally successful in convincing Congress of the advisability of restricting future appointments to officers trained in civil engineering. A law passed in 1906 indicated Congressional approval of this policy, and required that thereafter the Chief of the Bureau of Yards and Docks should be appointed from the personnel of the Corps of Civil Engineers.

Admiral Endicott's name came prominently before the Engineering Profession during the early years of his administration of the Bureau, when he took steps to change the then standard design of naval dry docks from timber to concrete and stone. He had clearly foreseen that wooden dry docks would not be able to meet the requirements of future battleships. Moreover, with their higher costs for maintenance and repairs they were not economical. The criticisms that were directed against him by those interested in the continued

construction of wooden dry docks were severe, and attempts were made in various ways to belittle his arguments and to influence his judgment and action, but in vain. His mind had been made up after careful study and consideration, and he remained steadfast in his beliefs. In the face of formidable opposition he convinced Congress of the soundness of his recommendations, which resulted in Congressional authority to change the construction of four dry docks, about to be started, from timber to concrete and stone. That was the last of wooden dry docks for Naval use, and the present standard of concrete and stone construction was thereby established.

Eleven dry docks were constructed under Admiral Endicott's supervision, two of which were steel floating structures. The first of the two floating docks, capable of docking a 16 000-ton vessel, was constructed at Sparrows Point, Baltimore, Md., and towed to New Orleans, La., where it has been in use up to the present time. The famous floating dry dock, *Dewey*, now in service in the Philippine Islands, was the second and the larger of the two. It was designed for docking a 20 000-ton vessel. The *Dewey* far surpassed in size any floating dock that had been constructed up to that time. It embodied some of the features of the New Orleans dock, which was patterned after an English design, but was so far improved and strengthened that the design can be termed an American one. These two docks have self-docking features which permit them to be repaired and painted without utilizing any other methods of docking.

In 1905, Admiral Endicott was appointed by President Roosevelt and confirmed by the Senate as the Navy Member of the Panama Canal Commission. This Commission made its headquarters in Washington, and he continued to perform this duty, in addition to his other work, until 1907, when the Commission was abolished and a new one appointed which made its headquarters and functioned at the Canal Zone in Panama.

Admiral Endicott's efforts were always directed to securing the greatest possible economy in the expenditure of public funds. He advocated and finally obtained the approval of Congress for the consolidation of all power plants at each of the larger Navy Yards into a central station. This resulted in a large saving of Government funds. He originated a design for large floating cranes with a lifting capacity of 100 gross tons and capable of handling guns, turrets, boilers, and other heavy objects.

Following his retirement, Admiral Endicott continued to make his home in Washington, D. C. In 1914 and again in 1917, he was called back to active duty with the Department of Justice, in connection with suits against the Navy Department.

On October 12, 1917, during the World War, he was again ordered to active duty and detailed to the Bureau of Yards and Docks. In addition to various technical duties assigned him, he acted at different times as President of four Naval Examining Boards to examine candidates for appointment to the Corps of Civil Engineers. One of these boards for the appointment of reserve engineers handled and rated the papers of more than 7 000 applicants.

In 1918, he was ordered to Norfolk, Va., for temporary duty as a member of a board to report on the extension of the harbor line at the Naval Operating Base at Hampton Roads, Va. He was later appointed a member of the Board created to recommend those deemed worthy of the award of the Congressional Medals of Honor, the Distinguished Service Medals, and the Navy Crosses, provided for in the Act of Congress approved February 4, 1919. He was relieved from active duty on June 30, 1920. The following is quoted from a letter of commendation given to Admiral Endicott by Secretary of the Navy Daniels for his war service:

"He performed exceptionally meritorious service in a duty of great responsibility, acting in an advisory capacity to the Chief of the Bureau of Yards and Docks, and as a member of various special boards dealing with matters of great importance in connection with the prosecution of the war."

Admiral Endicott was married in Dresden, Ohio, on May 29, 1872, to Elizabeth Adams. He is survived by his widow and seven daughters. His home life was exceedingly happy. He was a devoted husband and father. He was active in church circles, having served as a Vestryman of the Church of the Epiphany (Protestant Episcopal) of Washington. He was a member of the Theta Xi Fraternity; a Trustee of the Protestant Episcopal Church Home of Washington; a life member of the New Jersey Historical Society; a member of the Cosmos Club and Monday Evening Club of Washington; and an Honorary Member of the Washington Society of Civil Engineers.

His health had always been unusually good until within a year and one-half of his death and his tall, striking figure was a familiar and always welcome sight on the streets of Washington to his many friends. A heavy cold that he contracted in the latter part of February, 1926, developed into pneumonia and after a few days' illness, he quietly passed away on March 5. Burial was in Arlington National Cemetery on March 8, 1926, where full military honors were accorded him.

In the death of Admiral Endicott the Society has lost an honored member, who cherished a deep interest in its welfare. He was a Naval Officer and Engineer whose patriotism, professional attainments, and high ideals made him one of the outstanding public servants of his day. He always had the courage of his convictions and stood ready to fight if necessary for what he believed to be right. He never temporized where a question of principle was involved, and never took a backward step from the course he determined that he should follow. He has been well called "The Father of the Corps of Civil Engineers." He was an earnest and true friend, wise in counsel, charitable in judgment, kindly, with sympathy for all who appealed to him, and universally beloved by all whose privilege it was to know him.

Admiral Endicott was elected a Member of the American Society of Civil Engineers on April 4, 1877, retaining his connection with it for a period of forty-nine years, lacking one month. He served as a Director from 1901 to 1903; as Vice-President in 1908 and 1909; as President in 1911; and on the Board of Direction, as Past-President, from 1912 to 1916.

HENRY FLEETWOOD ALBRIGHT, M. Am. Soc. C. E.*

DIED MAY 11, 1926.

Henry Fleetwood Albright was born at Lancaster, Pa., on October 5, 1868. He received his education in the public schools of Philadelphia, Pa., and shortly after his graduation went to work as an office boy with the Union Pacific Railway Company in that city.

In 1892, while employed as a Sales Engineer for the Thompson-Houston Company, he attracted the attention of the Western Electric Company when he secured a contract from three of its best salesmen for an electric lighting plant for Wheeling, W. Va. The latter Company appreciating the efficiency of this transaction offered Mr. Albright the position of Sales Engineer which he accepted.

In 1894, he was transferred to the Construction Department of the Western Electric Company, in New York, N. Y., where as Supervising Electrical Engineer he designed and supervised the installation of electric power and lighting systems. His work was done in such an efficient manner that in 1897 he was promoted to the position of Factory Engineer, in charge of the design and supervision of the installation of electrical and mechanical building equipment, such as electric current generation, distribution for power and lighting, heating, sprinklers, plumbing, and elevators. He also supervised the installation of manufacturing equipment, such as machine tools, benches, and storeroom equipment in the New York Shop of the Company.

From 1898 to 1899, he served as Assistant Shop Superintendent and was in responsible charge of preparing layouts of manufacturing operations, including the routing of materials through the shop, as well as layouts of manufacturing equipment and store rooms.

From 1899 to 1908, Mr. Albright held the position of Shop Superintendent and his talent for systematic organization soon made the New York factory a model of efficiency. Realizing the probable rapid growth of the Company, he began to develop his organization along functional lines, grouping together, under responsible heads, departments that performed the same general function.

In 1908, Mr. Albright was made General Superintendent in charge of all manufacturing operations of the Western Electric Company in the United States. It was at this time that he moved to Chicago, Ill., where he made the Hawthorne Works, then being erected, his headquarters, and began the process of concentration of the Company's principal manufacturing activities at that plant.

The Hawthorne Works which is one of the largest and most complete manufacturing plants in the United States is the greatest testimonial to Mr. Albright's business ability. The first buildings at this plant were erected in 1905 and he supervised the layout for the Telephone Apparatus Shops and also an ultimate layout for the entire plant which has been consistently followed as various buildings have been added. During the period from 1908 to

* Memoir compiled from information of file at the Headquarters of the Society.

1923, the total floor space used at the Hawthorne Works for manufacturing was increased from 300 000 to 3 000 000 sq. ft. and the number of employees from 2 500 to 30 000. In the buildings of this plant, Mr. Albright installed the latest and best ideas in methods, equipment, and arrangement of machinery, thereby making the Works as a whole a model in factory building.

In addition to his duties for the Western Electric Company in the United States, Mr. Albright was also in responsible charge of the selection of sites and the design of buildings and manufacturing layouts for the ultimate development of properties of its foreign and affiliated Companies at Antwerp, Belgium, London, England, Paris, France, and Tokyo, Japan.

In 1923, Mr. Albright was made Vice-President and Director of the Western Electric Company, Incorporated, which position he held until his death. In this position he retained his duties as General Superintendent in charge of the manufacturing operations of the Company and in addition he was placed in charge of the selection of a site for a new manufacturing plant at Kearney, N. J., the design of the buildings and the development of manufacturing equipment layouts for that plant.

He died on May 11, 1926, at the Memorial Hospital in New York, N. Y., after a long illness. His body was taken to his home in Oak Park, Ill., and buried in Glen Oak Cemetery. On the day of the funeral the Hawthorne Works were closed as a mark of respect to Mr. Albright.

On December 26, 1892, Mr. Albright was married to Laura Heston, of Philadelphia, Pa., who, with a son, Henry F. Albright, Jr., and a daughter, Laura, survives him.

Relative to his ability as an engineer and executive and to his character as a man, Charles G. Du Bois, Chairman of the Board of Directors of the Western Electric Company, and a close business associate of Mr. Albright, has written the following appreciation:

"Mr. Albright was a man of unusual and marked abilities. He did his work carefully and completely. He envisioned great projects but he never overlooked or despised the details. As his responsibilities grew he trained others to take care of the lesser things and he trusted them. He was vigilant in the Company's interest on the daily work, but his mind constantly turned toward its future operations and the preparation to be made for them. He is gone but of no man can it be more truly said that his work lives after him.

"And yet it is not the superb and enduring work he left behind which has filled my thoughts of him in these past few weeks. It is his personality—the man himself apart from the work he did.

"Nowhere did he reveal himself to me so fully as in our travels together. His capacity for absorbing information on any subject, the wide range of his intellectual curiosity, and his remarkable memory made him always a most interesting traveling companion. And the mere being in motion on ship or train seemed to stimulate his memory of other scenes and incidents or stories so that he entertained his fellow-travelers much more I think than he supposed. For although completely fearless, he was naturally shy and self-effacing, usually concealing his inner feelings and covering with a mantle of scientific detachment and coolness a deeply emotional nature. He was a true scientist, unswerving in his search for facts and their meanings, but a discriminating taste, a spirit of artistry, and a love of beauty in every form colored and animated his outlook. This combination of qualities would have made him a

rare personality but he added to them an affection for and appreciation of his friends and associates which inspired in them a devotion to him such as few men ever achieve."

He was a member of the Institute of Electrical Engineers, of London, England; a member of the American Society of Mechanical Engineers and a Fellow of the American Institute of Electrical Engineers. He was also a member of the Union League Club, the Lotus Club, and the Oak Park Country Club of Chicago. He was a member of the Presbyterian Church.

Mr. Albright was elected a Member of the American Society of Civil Engineers on December 14, 1925.

JAMES WORK DEEN, M. Am. Soc. C. E.*

Died July 27, 1926.

James Work Deen was born on August 4, 1852, in West Waterford, Pa., the son of James and Rosanna (Work) Deen. He was a direct descendant of Richard Stone, Private in Captain Henderson's Company, Colonel Harmer's Regiment, First Battalion, Pennsylvania Continental Line. His education was acquired in the public schools of Pennsylvania and in the State Normal School.

Mr. Deen entered railway service as Rodman and Levelman on location of the Fulton and Franklin Railroad in 1877. The following year he was Instrumentman on the construction of the Selinsgrove and North Branch Railroad, and from April, 1879, to April, 1881, he was, successively, Rodman and Instrumentman, on Maintenance of Way, Middle Division of the Pennsylvania Railroad.

In 1881 he left the service of the Pennsylvania Company and joined the Engineering Corps of the Denver and Rio Grande Railway Company, with which Company he was thereafter continuously employed until the time of his retirement in 1923, a period of 42 years. One of Mr. Deen's earliest works of responsibility, during practically a lifetime of exacting railway engineering in the mountains, consisted of rebuilding 20 miles of line in Grape Creek Canyon, destroyed by the flood of 1881. His next notable position of trust lay in the rectification of alignment on Marshall Pass and in the Black Canyon of the Upper Gunnison River.

Afterward, in 1886, his remarkable talents for location were recognized in his appointment as Locating Engineer, and a year later he was placed in charge of construction on the Eagle River Extension, now a part of the main line between Denver, Colo., and Salt Lake City, Utah. In the winter of 1887-88, Mr. Deen was in charge of widening the line from Pueblo to Trinidad, Colo., to standard gauge, and early in 1888 was appointed Division Engineer of the Second and Third Divisions of the Denver and Rio Grande Railway, the former including the lines on the Marshall Pass route between

* Memoir prepared by Arthur O. Ridgway, M. Am. Soc. C. E.

Salida and Grand Junction, Colo., and the latter the lines on the Tennessee Pass route between Salida and Aspen, Colo.

It was during Mr. Deen's 25 years of service in this capacity that his most effective and lasting work as an engineer was accomplished. On the location in 1889 and in charge of the construction in 1890 of the standard gauge line over Tennessee Pass and through the Eagle and Colorado River Canyons, his ingenuity and engineering ability were brought into full play, and the results accomplished still bear witness to the remarkable ability of the man.

Mr. Deen never left the mountains in which he had spent the greater part of his life and to which he became firmly attached. Upon his retirement from active service he continued to reside at Salida, a respected citizen of the community.

In June, 1891, Mr. Deen was married to Martha J. Snyder, of Mifflintown, Pa., who, with two of his sisters, Mrs. Lillie D. Mosgrove, of Salida, and Mrs. C. P. Lantz, of Washington, D. C., survives him.

For thirteen years he served as a member of the Salida City Council, and always took more than an ordinary interest in civic, State, and National affairs. As a mark of honor, all public and business institutions of Salida were closed during the funeral ceremonies. No better tribute can be paid to his memory than that of the *Salida Mail* in its issue of July 30, 1926:

"The average citizen is an asset to a community while he lives in it, and a loss to it when he moves away or answers the final call. But it is given to some men to be of special value to the town and State in which they live. J. W. Deen was one of these. He was one of Salida's best citizens and one of the builders of the State."

He was a member of the Colorado Society of Engineers, Sons of the American Revolution, the Salida Chamber of Commerce, and an Honorary Member of the Salida Lions Club.

Mr. Deen was elected a Member of the American Society of Civil Engineers on January 6, 1892, and became a Life Member in 1925.

CHARLES GLEASON ELLIOTT, M. Am. Soc. C. E.*

Died September 14, 1926.

Charles Gleason Elliott was born in Lowell, Ill., on June 8, 1850, the son of John B. and Elizabeth (Searles) Elliott. He was educated at Illinois Wesleyan University, Oberlin College, and the University of Illinois, receiving the degree of Bachelor of Science from the latter institution in 1877, and that of Civil Engineer in 1893.

Following his graduation Mr. Elliott engaged in general engineering practice, principally in drainage engineering in the North Central States, and in geologic investigations in the oil fields of the West.

In 1884, he was appointed Sanitary Engineer by the Superintendent of the Hospital for the Insane at Indianapolis, Ind., to investigate and report

* Memoir compiled from information on file at the Headquarters of the Society.

on a plan for the drainage and sewerage of that institution. In 1886, he served as Chief Engineer for the Red River Valley Drainage Commission in charge of the surveys and design for the drainage of 809 000 acres in Minnesota.

From 1887 to 1889 he held the position of Engineer in charge of Special Drainage District No. 1 of Onarga, Douglas, and Danforth Townships, Iroquois County, Illinois, on the drainage of 16 000 acres of land. From 1890 to 1901, he was engaged in private practice as Engineer for the drainage of large farms and for drainage districts organized under the laws of Illinois. In 1896, he prepared the first *Bulletin* on agricultural drainage issued by the United States Department of Agriculture.

In 1902, Mr. Elliott entered the Government service as Drainage Expert and, later, was made Chief of Drainage Investigations, in the Office of Experiment Stations, Department of Agriculture, where he continued to carry on the work until 1913. He was commissioned by the Secretary of Agriculture in 1908 to visit Northern Europe and to investigate and report on land drainage, with particular reference to the drainage of turf and peat lands in Holland, England, and Germany. He also made investigations and reported on plans for Minnesota drainage, and, in addition, reported on the drainage of prairie lands in Canada. During the period of his Government service Mr. Elliott personally developed the methods now generally used in the arid regions of the West for draining irrigated lands, and thus saved many irrigation projects from apparent failure.

After 1913, he was engaged in private practice as a Consulting Drainage Engineer and, at the time of his death, was a member of the Elliott-Harman Engineering Company, with offices in Peoria, Ill., Memphis, Tenn., and Washington, D. C.

Mr. Elliott was the author of "Practical Farm Drainage" (1882, 1908), and "Engineering for Land Drainage" (1903, 1911, revised in 1919), both of which have been widely used as textbooks; also of "Drainage of Farm Lands" (*Bulletin 187,* U. S. Department of Agriculture, 1904). At the meeting of the International Engineering Congress in San Francisco, Calif., in 1915, he presented a paper on "Drainage as a Correlative of Irrigation".*

Mr. Elliott was a pioneer in agricultural drainage and for many years was recognized as one of the leading American authorities in that branch of engineering. He was a Republican in his political sympathies, and a member of the Illinois Society of Engineers and the Washington Society of Engineers.

He was married on January 1, 1879, to Lura M., daughter of Leonard L. Bullock, of Normal, Ill., and is survived by a son, Herman R. Elliott, of Montrose, Colo.; a daughter, Mrs. P. B. Morehouse, of Washington, D. C.; and a sister, Mrs. Ida Knapp, of Deerwood, Minn.

Mr. Elliott was elected a Member of the American Society of Civil Engineers on September 3, 1890.

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* *Transactions*, International Eng. Congress, San Francisco, Calif., Paper No. 39, Waterways and Irrigation, p. 510.

FRANCIS EDWIN HOUSE, M. Am. Soc. C. E.*

Died April 3, 1926.

Francis Edwin House, the eldest of a family of four sons of Henry A. and Mary E. (Goff) House, was born at Houseville, N. Y., on November 15, 1855. The family was of New England ancestry, and had founded the village early in the Nineteenth Century.

After a college preparatory course at a private school in Rochester, N. Y., Mr. House entered Rensselaer Polytechnic Institute, at Troy, N. Y., where he studied Civil Engineering and Chemistry, but was not graduated. He was a member of the Delta Phi Fraternity.

He left Rensselaer in 1877, in his Senior year, and went to Austin, Nev., then one of the important silver mining camps of the West. Here, for three years, he worked as Mining Engineer and Assayer for the principal mine of that camp. In Austin he also began the work with which he was to be identified for the remainder of his life—railroad engineering. Toward the end of his stay in Austin, in addition to his mining work, he completed the surveys and engineering work for the narrow-gauge mountain railway, connecting Austin with Battle Mountain, Nev., ninety miles distant.

In 1880, Mr. House entered the employ of the Chicago, Milwaukee and St. Paul Railway Company, and during the following three years was engaged in preliminary and location surveys for that railroad, the Wabash, and others, on extensions of their Systems in Missouri, Iowa, and Nebraska. He was made Division Roadmaster of the Chicago, Milwaukee and St. Paul Railway in November, 1883; General Roadmaster in 1887; and Trainmaster of the Kansas City Division in 1890.

In 1891 Mr. House joined the Engineering Staff of the Lake Shore and Michigan Southern Railway Company, and was engaged on construction for that System until in 1892 he went to the Pittsburgh and Lake Erie Railway Company as Engineer of Maintenance of Way. In 1894 he was promoted to the position of Chief Engineer.

In 1896 Mr. House went, with his superior, to build the Butler and Pittsburgh Railroad for the Carnegie Steel Company, and shortly afterward became Chief Engineer of the entire Pittsburgh, Bessemer and Lake Erie Railway, having charge of all improvements on the old line, as well as the construction of the new part, which was notable on account of the heavy work required in its construction.

The late Andrew Carnegie, then active head of the Company, was much pleased with Mr. House and his work, and his advancement was rapid. He was made General Superintendent of the road in 1897, and General Manager in 1901. His superior officer in the organization at that time writes:

"If the Carnegie Steel Company had continued its independent operation, I think he would have gone up in that organization in some other department than its railroad."

* Memoir prepared by a Committee of the Duluth Section, consisting of Messrs. W. A. Clark, W. H. Hoyt, and O. H. Dickerson, Members, Am. Soc. C. E.

Upon the formation of the United States Steel Corporation in 1901, Mr. House went to Duluth, Minn., as President and General Manager of the Duluth and Iron Range Railroad Company, which position he held continuously until his death, on April 3, 1926, except that during Federal control of the railroads under the United States Railroad Administration, he served as Federal Manager at the same time of both the Duluth and Iron Range Railroad, and the Duluth, Missabe and Northern Railway.

He was endowed with a splendid physique, and as an executive showed remarkable energy and great capacity for work, a contagious, almost boyish enthusiasm, and a very high ideal of justice and fair dealing. Although a believer in strict discipline, justice was tempered with mercy in his decisions, and their fairness was seldom questioned.

Mr. House was a remarkably fine type of man and citizen; devoted to his home and family; a loyal friend and a delightful associate; liberal in his support of worthy causes; of broad interests; keen in business; and remarkably successful as an engineer and executive. He served as an Elder in the Presbyterian Church for more than twenty-five years, and was active in its work. For nearly eighteen years he was a Director of the Duluth Young Men's Christian Association, for ten years its Vice-President, and its Acting President for two periods, amounting to about three years. He was also a member of the National War Work Council of the Association during the World War.

In 1918 his Alma Mater conferred on him the Honorary Degree of Master of Civil Engineering. Mr. House is the first and only person to have received it.

Mr. House was married in 1878 to Mary V. McCracken, at Des Moines, Iowa. His wife and four of six children survive him.

Mr. House was elected a Member of the American Society of Civil Engineers on May 1, 1895, and although for the last thirty years of his life he was not directly engaged in engineering work, he retained an active interest in engineering activities. The Duluth Section of the Society is greatly indebted to him as a leader in its organization and its first President.

AUGUSTUS SAYRE KIBBE, M. Am. Soc. C. E.*

DIED AUGUST 21, 1926.

August Sayre Kibbe was born in Albany, N. Y., on August 8, 1865, the son of Augustus Kibbe, whose ancestors came from Exeter, England, in 1611, and Sarah A. (Sayre) Kibbe, of French descent, whose family came from Rouen, France. As a boy he lived in Brooklyn, N. Y., and attended the Polytechnic Institute of Brooklyn, later going to Troy, N. Y., where he received the degree of Civil Engineer from the Rensselaer Polytechnic Institute in 1886.

Upon graduation Mr. Kibbe was Assistant to the Director of the Rensselaer Polytechnic Institute for one year, after which he became Assistant

* Memoir prepared by George H. Binkley and J. Otis Burrage, Members, Am. Soc. C. E.

Engineer with the State Engineer of New York, in charge of the Champlain Canal enlargement, with headquarters at Fort Edward, N. Y. After two years on this work he was appointed Special Assistant to the New York State Engineer and Surveyor in August, 1888, with headquarters at Albany, N. Y., and was engaged in a number of important undertakings in connection with State surveys, water supply, and regulation, including the Black River and Canal water supply, the Genesee River water supply and storage, Adirondack reservoirs, New York maps for the Municipal Consolidation Commission, New York-Canada and New York-Pennsylvania boundaries, etc.

In April, 1892, Mr. Kibbe resigned from the New York State Engineering Department to engage in electric railway work as Engineer and Superintendent of the Woodbridge and Turner Engineering Company, in New York, N. Y. While with this firm he designed and constructed a number of electric railways, among which were the Paterson Central Railroad of New Jersey, the Washington, Alexandria and Mount Vernon Railway, the Brigantine (N. J.) Transit Company's Railway, the Chester (Pa.) Traction System, and parts of the People's Traction Company of Philadelphia, Pa. He severed his connection with the Woodbridge and Turner Engineering Company in October, 1895, to become Engineer of Construction of William Wharton, Jr., and Company, with which Company he remained for four years. While with this Company he built the Fairmont Park Transportation Company's Railway, Philadelphia.

Commencing in June, 1899, as Engineer for the American Railways Company, Philadelphia, Mr. Kibbe became Chief Engineer of that Company in 1904, which position he held until November 30, 1910. During this time he had engineering charge of the construction, equipment, and operation of the electric railway, lighting, and power plants of the American Railways Company, which comprised about 350 miles of track and a capacity of 15 000 kw. in power stations, located in various cities of the Middle Western States. In 1911, he opened offices as a Consulting Engineer in Chicago, Ill., but in 1912, transferred his activities to Reno, Nev. From 1914 until his death Mr. Kibbe was with the Key Transit System, Oakland, Calif., as Consulting Engineer in charge of valuation and rates. During this time he also served as Consultant in valuation matters for the Northern Electric Company and the Sacramento-Northern Railway of California.

Outside his engineering work, Mr. Kibbe's great interest was in wild life conservation and he was Secretary of the California Societies for the Conservation of Wild Life. For six years previous to his death he served as President of the Audubon Association of the Pacific. Probably no bird lover was better informed as to California birds, and he was considered an authority both on the knowledge of birds and their protection, which was the purpose of the Association. He was also a member of the Sierra Club and one of the original members of the Pacific Railway Club.

Mr. Kibbe was always a student and a man of high ideals and character. He was quiet, modest, and retiring, but with his keen, analytical mind he arrived at his decisions and stood firmly for them in his unassuming manner.

He was loved and respected for his faithfulness, loyalty, and honesty by all with whom he came in contact.

He left a son, Augustus Sullivan Kibbe, of West Chester, Pa., and a daughter, Helen Towne, of Dayton, Ohio. His widow, Bessie W. Kibbe, resides in Berkeley, Calif.

Mr. Kibbe was elected a Junior of the American Society of Civil Engineers on March 6, 1889, and a Member on August 31, 1909.

WILBUR FISK McCLURE, M. Am. Soc. C. E.*

Died June 22, 1926.

Wilbur Fisk McClure was born in Perryville, Ohio, on December 13, 1856, the second son of Thomas Clarendon and Sarah A. (Stephens) McClure. His early childhood was spent in Ohio and Pennsylvania where his father held various charges as an ordained Minister of the Methodist Episcopal Church in the days when "itineracy" meant moving at least every two years. He attended the Southwestern Normal School at California, Pa., and on leaving this institution taught in the public schools of Fayette and Washington Counties, Pennsylvania, at Carthage, Mo., and in the Cherokee Nation.

While engaged as a teacher Mr. McClure took up the study of engineering, for which he left the Teaching Profession early in 1879, beginning work with the St. Louis and San Francisco Railroad Company as an Assistant Transitman on location. From November, 1879, to June 1, 1882, he was in the employ of the same Company as Assistant Engineer in charge of construction, or as Assistant on location in Missouri, Kansas, Arkansas, Indian Territory, and Texas. During the last six months of 1882, he was in charge of the Land Department of the Company, surveying lands and town sites.

In March, 1883, Mr. McClure went to Deming, N. Mex., as Assistant Engineer in charge of tracklaying on the Deming, Silver City, and Pacific Railroad (narrow-gauge). On the completion of this road in the same year he moved to Los Angeles, Calif., and from 1883 to 1886 was engaged in private practice, chiefly in surveying oil claims for the Pacific Coast Oil Company and acting as Engineer for the Mountain Water Company.

From 1887 to 1893, he served as Chief Engineer of the Los Angeles Terminal Railway System (now the Union Pacific in California). In 1893 he went into Mexico where he surveyed and reported on mining property for Mr. T. B. Burnett and others of Los Angeles. While on this work he contracted malarial fever and was brought home to Los Angeles, where he lay seriously ill for three months. The effects of this illness, in fact, never entirely left him.

While convalescing at his home in Los Angeles, Mr. McClure received a call to the Methodist ministry and for the next seven years he served as a lay missionary in that region in California lying mainly east of the Sierra Nevada,

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* Memoir prepared by Frank Adams and Paul Bailey, Members, Am. Soc. C. E.

with successive charges in Greenville, Plumas County; Bishop, Inyo County; Cedarville, Modoc County; and Truckee. At that time this was a pioneer, isolated field, and he is still affectionately remembered there for his sincere and kindly spiritual leadership.

As ill health had forced Mr. McClure out of engineering in 1893, so in turn it forced him out of the ministry and back to engineering in 1900. In that year he moved his home to Berkeley, Calif., where he remained for the next twelve years. From 1900 to 1905, he was Superintendent for Rudolph Axman and Robert Wakefield, Contractors, in the removal of Skagg, Arch, and Blossom Rocks to a depth of 30 ft. from San Francisco Bay. In 1905, he was elected City Engineer of Berkeley, serving in that capacity until 1909, when he became Commissioner of Public Works at the time Berkeley adopted the Commission Form of Government.

In 1911 he returned to private engineering practice, continuing in this until his appointment by Governor Hiram Johnson, on February 13, 1912, to the office of State Engineer of California. From that appointment dates his largest engineering service. Through three successive gubernatorial administrations and a service exceeding fourteen years Mr. McClure continued as State Engineer of California. From July 30, 1921, until his death, in addition to holding the position of State Engineer, he was Chief of the Division of Engineering and Irrigation of the State Department of Public Works, and from August 17, 1923, Director of that Department.

At the time of Mr. McClure's appointment by Governor Johnson in 1912 the jurisdiction of the State Engineer's Office related principally to work in and about State institutions, with minor enterprises in road construction and rectification of river channels. During the period in which he was State Engineer, however, the activities of the office underwent a great expansion. State supervision of irrigation and reclamation development and of dam construction originated and grew to large proportions. Engineering investigational work also became an important function of the office.

Between 1911 and 1915 the California Irrigation District Act was perfected, through many amendments, to the point where it became practical to proceed with substantial irrigation development. In 1913, the California Irrigation District Bond Commission (now the California Bond Certification Commission), composed of the State Engineer, Attorney General, and Superintendent of Banks, was created by the Legislature. Through the separate jurisdiction of the State Engineer's Office under the California Irrigation District Act and the jurisdiction of the Bond Commission, the State, since 1912, has supervised one of the greatest expansions in agriculture through irrigation that has occurred in any locality in the United States. This expansion has involved the organization of about 100 irrigation districts and the issuance of more than $120 000 000 of irrigation district bonds for construction work. The organization of these districts, and the development of their irrigation systems all passed under Mr. McClure's official review. The total area in these districts is 3 500 000 acres. This is a greater area than is at present irrigable under all the Federal irrigation projects combined.

The assemblage of the large volume of capital at low interest rates necessary to construct the works for irrigating this vast area was made possible through the successful administration of State supervision of irrigation projects. This successful administration has made irrigation securities readily purchasable by the buying public. Almost without exception, these entire areas have met their financial obligations as they became due.

This great and rapid expansion of the irrigated areas in California advanced conditions to the point where organization of large areas became necessary under conditions not contemplated by the California Irrigation District Act. Accordingly, the California Water Storage District Act was passed by the Legislature of 1921 and the California Water Conservation District Act in 1923. Under the supervision of the State Engineer as provided in these Acts, districts of about 2 300 000 acres have organized to irrigate new lands and to improve the water supply of other lands already under irrigation.

The great increase in productivity resulting from the successful irrigation of these large areas has enabled California to take a ranking position among the States of the Union in agricultural production. Without irrigation California would have reached its maximum production as long as forty years ago. At that time the greatest area profitable to cultivate by dry-farming methods had been brought into use. Through the last forty years, by the introduction of irrigation and without increasing the total area under cultivation, the value of agricultural production in the State has increased manyfold.

In order that progress might continue uninterruptedly and that agriculture and other industries dependent on water might further contribute to the prosperity of the State, engineering investigational work concerning available water supplies and their most advantageous utilization became one of the most important functions of the State Engineer's Office during the later years of Mr. McClure's administration. This work produced the reports on the water resources of the State presented to the Legislatures of 1923 and 1925. These reports contained the first complete inventories of the waters of California and of the future needs for water in order to accommodate the full development of the State's resources.

The culminating report to this work was under preparation at the time of Mr. McClure's death. This report will present to the Legislature of 1927 a plan for co-ordinating the development of the State's waters for all purposes, and for equalizing the naturally unequal distribution of such waters to the end that all sections of the State may receive their full quota.

Parallel with this immense program of irrigation development, the reclamation of overflow lands in the Sacramento and San Joaquin Valleys has also progressed rapidly under legislation passed in 1911. The State Engineer's Office has participated with the State Reclamation Board and the California Débris Commission, as provided in the legislation for the State supervision of these projects. All this work stands as a monument to the administration of Mr. McClure.

Possibly one of the most important engineering services rendered by him was as California member of the Colorado River Commission, which was

organized on January 27, 1922, under the Chairmanship of the Secretary of Commerce, Herbert Hoover, Hon. M. Am. Soc. C. E., and which practically concluded its work with the signing of the Colorado River Compact at Santa Fé, N. Mex., on November 24 of the same year.

In a letter to the writers, Secretary Hoover states that Mr. McClure's "unceasing devotion in the interests of the State of California was accompanied by a breadth of vision as to national interests which marked his character at all times," and "his engineering knowledge and familiarity with all the facts bearing on the most difficult questions and his fine personality made him one of the most important members of the Commission." The controversies that followed the signing of the Compact at Santa Fé were a constant source of concern to Mr. McClure, and to his death he continued to lend his influence toward a fair and equitable solution of what he recognized to be a problem of overshadowing importance to California and the entire Southwest.

If there is one characteristic that stands out above others in Mr. McClure's administration of the important office he occupied from 1912 to his death, it was his desire to perform his duties fully and faithfully and to act with absolute fairness and justice. He knew no greater professional pleasure than to help forward an enterprise he deemed to be worthy, and it was always with genuine regret that he withheld approval from one, sincerely undertaken, which he considered economically or physically unjustified. Although never seeking official authority, he did not hesitate to exercise that given to him by law, and would brook no subterfuge or dishonesty. His stand on matters before him was always based on his own ideas of right. It was his nature to side with the smaller and weaker if convinced they were honest in their endeavors, and because of his total lack of self-interest he was always able to inspire loyalty and the utmost effort among those working under him. The same fine qualities that endeared him to his family and those of his organization, endeared him also to all who came into friendly contact with him.

Mr. McClure was devoted to his family. He was first married at Candor, Pa., on August 17, 1882, to Sarah Hunter McCalmont, whom he met while attending the Normal School at California. There were four daughters of this marriage—Mrs. J. W. Mahoney, of Berkeley, Calif., Mrs. G. A. Robinson, of Merced, Calif., Miss Katherine McClure, of Sacramento, Calif., and Clara Clark McClure, who died in infancy. Following the death of his first wife in 1915, he was married on October 9, 1916, to Margaret Alter, a friend of his childhood, who survives him.

Not only was Mr. McClure devoted to his family, but he entertained the deepest feeling for his fellow men. Almost invariably he was an officer of his church, and both in Berkeley and in Sacramento, where he resided at the time of his death, he was also an officer in the Young Men's Christian Association. All worthy causes appealed to his kindly sympathy, and to these he was a consistent and conscientious tither.

Mr. McClure was elected a Member of the American Society of Civil Engineers on November 3, 1886, but resigned on December 31, 1894. He was elected a Member a second time on December 14, 1925.

CHAUNCY RUSCH PERRY, M. Am. Soc. C. E.*

DIED JUNE 24, 1926.

Chauncy Rusch Perry was born at Waltham, Mass., on May 6, 1872. He was the son of the Rev. John Philander and Emma (Rusch) Perry. His boyhood was spent mostly in New Ipswich, N. H., to which town his parents removed while he was a child. His preliminary education was obtained in the public schools of New Ipswich and the New Ipswich Academy of which institution he later became a Trustee.

In June, 1890, Mr. Perry entered the Engineering Department of the Boston Bridge Works where he remained for two years, acquiring experience in drafting and fitting himself for higher education. In September, 1892, he entered the Lawrence Scientific School and was graduated therefrom in June, 1895, with the degree of Bachelor of Science.

For the next three years Mr. Perry was employed by the Boston Transit Commission under Howard A. Carson, Hon. M. Am. Soc. C. E., first as a Draftsman and later as Assistant Engineer on the design of the steel framework of the Boston Subway. After an interval of eight years, during which he was with J. R. Worcester, M. Am. Soc. C. E., Consulting Engineer, and with the Boston Bridge Works, designing and estimating, he returned to the Transit Commission, by which he was employed as Assistant Engineer until 1910. During this time he was engaged in the design of many complicated parts of the Boston subways, tunnels, and stations, dealing not only with structural steel framework, tunnel shields, etc., but also with reinforced concrete sections.

For the greater part of the last sixteen years of his life Mr. Perry was connected with J. R. Worcester and Company, although at various times he undertook independent engagements. The latter included the design of the reinforced concrete of the Germanic Museum of Harvard University and of the Morgan Memorial in Boston; the design of the steelwork of the Providence Armory; a study of standard library stacks for the Library Bureau; buildings and crane tracks for the Hunt-Spiller Manufacturing Corporation, South Boston; the Squantum Destroyer Plant; Alameda Shipyards and Hog Island Plant for the United States Navy; Fore River Battleship Yard; and an exhaustive examination of bridges of the United Fruit Company in Costa Rica. He gave much thought to the design of tunnel shields, which were used in the construction of various tunnels.

Mr. Perry was married on June 21, 1896, to Helen Tuttle, who, with a daughter and three sons, survives him. He had few interests outside his home life and his work, although he was a violinist of ability and devoted what spare time he had to music. He was a persistent student and investigator, and delighted in solving complicated problems in his line of work. His clear reasoning and balanced judgment were relied on by his employers and associates when difficult questions were involved. He was a faithful

* Memoir prepared by J. R. Worcester, M. Am. Soc. C. E.

member of the New Jerusalem Church, and without ostentation endeavored to carry into actual life the teachings of his religion.

In disposition and bearing Mr. Perry was modest, deferential, and tolerant of the shortcomings in others, but firm in facing unpleasant situations, stoutly maintaining his side in an argument when convinced of its correctness. His nature was optimistic, always looking for the cheerful features in every occurrence. His mind was ever active, acquiring information of all kinds, which he was ready to share with others on every suitable occasion. Although connected with the Society for twenty-one years, he did not attend its meetings or take part in the discussions. He did, however, value highly its publications and made full use of the information derived therefrom.

Early in 1926 he suffered a severe attack of influenza from which he did not fully recover his strength. He kept at his work, however, until the day of his death, which occurred without a moment's warning on June 24, 1926.

Mr. Perry was elected an Associate Member of the American Society of Civil Engineers, on February 1, 1905, and a member on June 1, 1909.

HENRY CLARK THOMPSON, M. Am. Soc. C. E.*

DIED APRIL 13, 1926.

Henry Clark Thompson was born in Troy, N. Y., on September 26, 1861, the third son of John Calhoun Thompson, a Captain in the Civil War, who was the grandson of Daniel Thompson, a soldier of the American Revolution, and a lineal descendant of earlier settlers in America. His mother, whose maiden name was Anna Thompson, was also a member of an early American family, not however related in any way to the Daniel Thompsons.

In the early part of 1862, the City of Troy was partially destroyed by fire, and the Thompson family moved to New York, N. Y. At the age of eight, Henry Clark Thompson went to live on his father's country estate near Morristown, N. J., but returned to be graduated from a New York public school. From his father who was a builder, actively interested in the construction, at different times, of many houses in New York, Mr. Thompson inherited a taste for the structural work in which he, during the earlier years of his professional life, co-operated to some extent. Entering the employ of Maclay and Davies, Civil Engineers and Surveyors, of New York, the boy made good to an exceptional degree and, in his nineteenth year, was placed in charge of the building of the large iron pier at Rockaway.

At twenty years of age the Western country lured him as it did many other young men of the period, and he accepted an offer to serve as Assistant Engineer on the construction of a part of the new Denver, Western, and Pacific

* Memoir prepared by C. E. Chase, Asst. Div. Engr., M. of W. Dept., River Div., N. Y. C. R. R., Weehawken, N. J.

Railroad. When this work was well advanced, Mr. Thompson turned to another attraction of the same kind and contributed his share to the building of the road now famous as the Atchison, Topeka and Santa Fé. About that time an urgent appeal from his father brought him back East, the appeal taking the tangible and practical form of an offer to put the young engineer through college. He was graduated from the Columbia School of Mines in 1886. The Faculty statements, still preserved, of his progress as a student reveal that the high grades of "nine" and "ten" were not uncommon throughout his four years of attendance.

Immediately after graduation, Mr. Thompson secured a position as Assistant Engineer with the Suburban Elevated Railroad of New York. This, like his previous work, was construction. He spent fifteen months in this capacity, and then accepted a position as Resident Engineer of the Orange Mountain Land Company of New Jersey, on a \$500 000 plan, involving the layout and construction of streets, sewers, and water supply.

In 1889, Mr. Thompson had reached the point where he felt justified in going into business for himself. In the decade which followed, during a few years of which very little capital was risked in enterprise, he yet found more than enough to do. His work was varied, including highways, bridges, and pavements; water supply, sewers, and drainage; electric railway; and the erection of sixty or more buildings. Also during this period, as City Surveyor and as Engineer of the Village of Williamsburg, N. Y., he conducted surveys of farms and estates, laid out and monumented 50 miles of city streets and surveyed more than 10 000 city lots.

In 1900, Mr. Thompson became Supervisor of Bridges and Buildings for the New York Central Railroad Company (West Shore) at Weehawken, N. J., and shortly established a record for bridge building by completing more than 100 spans "under traffic" in 100 consecutive days. In 1902, he was appointed Division Engineer, his territory extending from Jersey City, N. J., to Albany, N. Y., and remained in resident charge of maintenance for twenty-four years, until the time of his illness and death.

He was a railroad and civil engineer of calm judgment and marked resource, as well as a man whose counsel was valued and sought. For himself, he desired no wider field of effort than to remain of service to his Company and its leading officers, and to set a good example for his subordinates to follow. Many of his co-workers, especially those who were nearest to him in the New York Central service, have felt very keenly the sense of personal loss.

About twenty years ago, Mr. Thompson was married to Mary Lawson Archer who, with their two children, a son and a daughter, survives him.

He was a Royal Arch Mason and a member of Eureka Lodge, New York. He also belonged to several railroad, engineering, and Columbia University clubs and societies.

Mr. Thompson was elected a Member of the American Society of Civil Engineers on February 4, 1903.

FREDERICK HOWARD TILLINGHAST, M. Am. Soc. C. E.*

Died July 15, 1926.

Frederick Howard Tillinghast was born in Providence, R. I., on September 19, 1877. At the age of twenty-one he was graduated from Brown University with the degree of Civil Engineer. The year following his graduation (October, 1899-June, 1900), he took a Post-Graduate Course in Hydraulics and Sanitary Engineering at the Massachusetts Institute of Technology.

Mr. Tillinghast's first experience in engineering was in railroading. He worked on the location, construction, and maintenance of more than one line in the Eastern States. Subsequently, he became connected with the United States Geological Survey as Hydrographer, making investigations of certain streams in New York State with the idea of an enlarged water supply for the City of New York.

In 1904, he joined the United States Reclamation Service and was with that service for a number of years. During this period he worked on the Belle Forche Irrigation Project in South Dakota, on the location, design, and construction of canals. He also revised the cost accounting system of the Project. After completing a few short assignments Mr. Tillinghast was placed in charge of the construction of the East Park Dam on the Orland Project, California. His most important work with the Reclamation Service, however, was on the Lahontan Dam, Nevada. He began with the preliminary field investigations, worked on designs, and finally became Resident Engineer on construction. When this work was finished he severed connections with the Service and engaged for a time in private practice. War conditions, however, made business uncertain and, in 1918, he took a position with the Sutter Basin Company in California, directing surveys. Later, he was promoted to be Resident Engineer, and, in 1921, was appointed Chief Engineer, which position he held until his death.

The circumstances connected with Mr. Tillinghast's death were more than ordinarily pathetic. About three years ago Mrs. Tillinghast began to fail in health and, in spite of the best medical advice, gradually became a hopeless invalid unable to help herself in the smallest degree. It seemed at this time as if few men were more needed than Mr. Tillinghast. His work occupied him during the day, and when that was done he went home to play the part of nurse. His own health during the last months preceding his death had not been the best, but there was apparently no occasion for alarm. However, the end came for him without a moment's warning. He is survived by his invalid wife and two sons, the elder of whom is married and lives in Woodland, Calif., while the younger has been attending the Junior College in Sacramento, Calif.

It always seems a futile thing to eulogize a man after he has gone, but, because Mr. Tillinghast was an Engineer and had certain gifts or acquirements which are desirable in a member of that profession, it may not be out

* Memoir prepared by S. H. Searancke, Assoc. M. Am. Soc. C. E.

of place to mention these, since they redound to his professional credit. He had the ability of mixing easily with all classes of people and was on terms of familiarity with the men he met in his work, whether superiors, equals, or subordinates. This gift of friendliness gained for him the regard and confidence of his fellow workers. He had in no small degree that plainness or commonness of manner which forms such a strong link of fellowship between a man in charge of work and those who work for him, and which almost always commands the highest type of loyalty. When he first went to the Sutter Basin in a more or less subordinate position, he was familiarly known as "Tilly", and it is significant of the man's own attitude that, after his appointment as Chief Engineer, no one thought it necessary to call him by any other name.

He had also the faculty of organizing men and directing work. He had confidence in those who worked for him, and knew how to shift part of the responsibility from his own shoulders to the shoulders of his subordinates, allowing them to find their own solutions to their problems.

For his co-workers, fellow-members in the Society, and a host of others who knew and liked him, there is much sadness in the thought that a friend has departed and will never be heard from again. There is nevertheless a certain satisfaction in the opportunity to sketch thus briefly the life of a plain engineer, not the type of man who has made for himself a nation-wide reputation, or who has had big work named for him, but the type that is never advertised and rarely even mentioned outside the small sphere of his own activities—one who has gone about his work in a modest and conscientious manner, giving much thought to his profession, and but little thought to the gratitude or rewards that might be due him.

Mr. Tillinghast was elected an Associate Member of the American Society of Civil Engineers on May 1, 1907, and a Member on April 1, 1914. It is interesting to note that among those referred to in his application are the names of A. P. Davis, and John R. Freeman, Past-Presidents, Am. Soc. C. E., and F. H. Newell, M. Am. Soc. C. E. Mr. Tillinghast was a member of both the Sacramento and San Francisco Local Sections.

JAMES HOLLIS WELLS, M. Am. Soc. C. E.*

Died September 24, 1926.

James Hollis Wells was born in England on August 3, 1864, the son of Arthur and Charlotte (Hollis) Wells. His grandfather, Richard Hollis, was an officer in the King's (1st) Dragoon Guards and fought under Wellington in the Battle of Waterloo. Later, he was Captain and Adjutant of the Grenadier Guards

The family came to America when James Hollis Wells was still a boy and settled in Bethlehem, Pa. He received his education in the public schools of Bethlehem, and in his early manhood, taught in the Grammar Schools of

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* Memoir prepared by James Dougan, Assoc. M. Am. Soc. C. E.

South Bethlehem. He was graduated and received the degree of Civil Engineer from Lehigh University in 1885.

While attending the University, from 1883 until after his graduation, Mr. Wells was connected with the Borough Engineering Department of Bethlehem as Assistant Engineer, and with the Survey Department of the Philadelphia and Reading Railroad Company. From 1886 to 1889, he was with the Department of Public Works of New York, N. Y., as Assistant Engineer and Inspector of asphalt and other paving. From 1889 to 1891 he served as Engineer for J. D. and T. D. Crimmins, General Contractors, and in 1892 and 1893, he was Clerk of the Works for Cornelius Vanderbilt and Engineer for the firm of Renwick, Aspinwall and Russell, Architects, on its building work.

In 1893 Mr. Wells became associated with Mr. W. H. Russell. This partnership was afterward merged into the firm of Clinton and Russell, and, later, that of Wells, Holton and George, Architects. At the time of his death, Colonel Wells was the Senior Member of the firm.

He was well versed and quite active in various lines of engineering, and due to his close affiliation with the building industry was considered an expert in all questions of engineering, foundations, structural steel design, plumbing, heating, power-construction details, methods of building, and similar work. Among the prominent buildings with which he was identified, and in which unusual conditions were encountered, were the Hotel Astor, Hudson Terminal Buildings, Whitehall Building, Broad Exchange Building, Apthorp Apartments, Graham Court Apartments, Mutual Life Buildings, 7th and 71st Regiment Armories, and Mecca Temple in New York; Imperial Oil Building, Toronto, Ont., Canada; Humble Oil Building, Houston, Tex.; Whitney Central Bank Building, New Orleans, La.; Mutual Life Insurance Building, Cape Town, South Africa; and the Mutual Assurance Building, Life Insurance Company of Virginia Building, and National State and City Bank Building, Richmond, Va. In addition to his activities with his own firm, he was Advisory Architect of the Elks National Memorial Building, Chicago, Ill.

Colonel Wells was also appointed by Governor Woodrow Wilson of New Jersey in 1913 as a member of the New Jersey Bridge and Tunnel Commission, and served as Consulting Engineer of the United States Treasury Department from 1914 to 1918, on the foundations of the Treasury Building in New York, and the engineering and construction features of the Department of the Interior Building at Washington, D. C.

He played an important part in the business world of both New York and New Jersey. He was a Director of the Trust Company of New Jersey, a member of the Executive Committee and Chairman of the Auditing Committee of that Company. He was also President of the Realty Company of New Jersey, and a Director of the Colonial Life Insurance Company of America.

Colonel Wells inherited a love of military service from his distinguished grandfather and served for more than thirty years in the National Guard of New York. He began his military career in the Fifth Company, 7th Regi-

ment, N. Y. N. G., in 1892, was commissioned Second Lieutenant, Company F, 71st Infantry, in December, 1892; Captain, in 1893; Major, in 1898; Lieutenant Colonel, in 1901; and Colonel, in 1917. He saw service with the 71st Regiment in Cuba as Major and commanded part of the regiment in the Battle of San Juan Hill on July 1, 1898. Later, he became Engineer Officer of the 1st Division, 5th Army Corps in Cuba.

He was a member of the American Society of Mechanical Engineers; the Engineers' Club of New York; a member of the Board of Governors and Chairman of the House Committee of the Railroad Club of New York; member of the Carteret and Union League Clubs, of Jersey City, and the Westmoreland Club, of Richmond, Va.; Past-Master, Bunting Lodge, F. and A. M.; member of Mecca Temple, A. A. O. N. M. S., Scottish Rite Masons; Benevolent Protective Order of Elks, Jersey City Lodge No. 211, and many engineering and military societies. He had been Vice-President of the New York State Rifle Association and a Director of the National Rifle Association of the United States, and President of the National Guard Association of the State of New York. He was also Adjutant of the United States Rifle Team that won the Palma Trophy at Bisley, England, in 1903.

In April, 1892, Colonel Wells was married to Belle Porter White. They have one son, Thomas Richmond Wells. The family residence is in Jersey City, where Colonel Wells was prominent in social and civic affairs.

He was buried in New York Bay Cemetery, Jersey City. Among the honorary pall-bearers were former Governor John K. Tener, of Pennsylvania, Gen. William C. Heppenheimer, and Joseph T. Fanning, Past Grand Exalted Ruler of the Elks. Fifty-six officers from the 71st Regiment, New York, who were formerly under the command of Colonel Wells, attended the funeral services, as well as many representatives of the building industry.

Colonel Wells was elected a Member of the American Society of Civil Engineers on February 5, 1901.

THOMAS PENGELLY ELLIS, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 27, 1920.

Thomas Pengelly Ellis was born at San José, Calif., on October 22, 1884, but received his preliminary education in Canada. Removing with his father to Denver, Colo., he entered the Colorado School of Mines, from which he was graduated in 1907 with the degree of Engineer of Mines.

While attending the School of Mines, Mr. Ellis was engaged as Field and Office Assistant in the Engineer's Office of the Denver Tramway Company on the design and layout of electric sub-stations, curves, and switches. He was also engaged as Transitman and Assayer in the mines of the Cripple Creek and Clear Creek Districts, Colorado.

After his graduation, Mr. Ellis spent a year in the State of Washington on the sub-division of a portion of Mercer Island, Lake Washington, and as

* Memoir prepared by William S. Post, Assoc. M. Am. Soc. C. E.

Contractor's Engineer in re-grading work in Seattle. After a few months of appraisal work for the Denver Union Water Company, Denver, Colo., and in mining and geological work in Clear Creek County, in March, 1908, he entered the employ of the Bureau of Engineering of the City and County of Denver, as Draftsman on the design and assessments of street improvements, sewers, etc. In March, 1910, he was appointed Inspector of Construction on part of the sewerage system, and in October of that year was made Division Engineer in charge of highway improvements, which position he retained until April, 1911. The occasion of his retirement from the Bureau of Engineering on a question of ethics was greatly to his credit.

In 1911, Mr. Ellis removed to San Diego, Calif., and until December, 1917, was connected with the Volcan Land and Water Company and the Cuyamaca Water Company, as Office Engineer, Field Engineer, Engineer of Land Department, and Valuation Engineer.

From February, 1918, to April, 1920, he was engaged as Resident Engineer for the Southern Division of Shipyard Plants of the United States Shipping Board as Supervisor and Inspector.

In April, 1920, Mr. Ellis became a partner of Mr. D. A. Dickie in the Dickie Construction Company and was engaged in salvaging piling at the Liberty Plant at Alameda, Calif. In the midst of these activities, he died from pneumonia on September 27, 1920. He is survived by his widow and two children.

Mr. Ellis was a man of great charm, high technical ability, and held steadfastly throughout his career to the ethics of the Engineering Profession. In his young manhood he had served as an artilleryman in the Canadian Militia and retained throughout his life an alert military carriage. Much to his regret the development of heart trouble prevented him from offering his services to his country in the World War. As a citizen of San Diego, he was highly thought of by the community.

Mr. Ellis was elected an Associate Member of the American Society of Civil Engineers on May 13, 1918.

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